THE

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GLOSSARY OF INDIAN WORDS.

Bajri—Coarse gravel.
Band—An earthen embankment.
Boldar—A member of the regulating establishment.
Chaks—A block of land.
Cheri—A kind of grain.
Cholum— Do.
Cumbu— Do.
Cusec—Cubic feet per second.
Doab—A tract of land between two rivers.
Ghani—Oil mill, worked by bullocks.
Jamadar—Head of regulating establishment.
Jhil—Lake.
Juar—A kind of grain.
Kalarathi—Sour or "reh" soil.
Kankar—Nodular hydraulic limestone.
Karrie—A baulk of timber.
Khadir—Low land adjacent to river.
Kharif—Wet weather crop.
Kiaris—Compartments into which fields are divided for irrigation.
Lakh—1,00,000.
Mahout—An elephant-driver.
Malguzari—Land-tax.
Maund—A weight equal to 82⅔lbs.
Mhot—Bullock water-lift.
Naddi—A river or stream.
Nullah—A drain or drainage channel.
Pargana—A sub-collectorate.
Picottah—Manual water-lift.
Pucca—Strong, substantial.
Rabi—Cold weather crop.
Ragi—A kind of grain.
Rau—Torrent.
Sal—A kind of timber.
Surkhi—Pounded brick.
Taluk—A sub-collectorate.
Tatils—Rotational closure of canal channels.
Zilladar—A canal revenue subordinate.

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PAPER No. 1.

Smart's Shutters.

In the Tanjore Delta (and of course elsewhere) it was found that channels taking off from a river with sandy bed at a weir became choked with sand up to a certain fixed level. If the sand was removed it was very quickly replaced. If it was not removed it did not increase except slowly. If the weir was raised the level of the sand rose; and it was obvious in fact that the weir was the cause of the sand accretion. The cure was to substitute for the weir something which could be removed during freshes, when the sand was principally in motion, and replaced at other times so as to hold up the water to any desired level. Small shutters were objectionable as being troublesome to work, while the numerous piers formed an obstruction almost as great as a weir. The outcome of numerous experiments led to the adoption of what are known as "Smart's" shutters, the credit of which is due to Colonel Smart, R.E., now retired. The type evolved was similar to the model (Plates 1 and 8), but the rollers ran on discs strung on the roller axle, the same method was employed for the sprocket wheels, the rollers were bushed with delta metal, and the pins and discs were of delta metal. These worked very well in the shops and the co-efficient of friction was stated to be even less than that of Stoney's gearing. On works they did not do so well. The use of delta metal was first abandoned as being unnecessary. The discs clogged with mud and dust and did not revolve and they also were abandoned, the final type being as in the model. These shutters have been erected of all sizes up to 40 feet × 12 feet and there are now many in use in the Madras Presidency.

The advantages of a system of large shutters are patent. The régime of a river is practically not interfered with, since the obstruction caused by the piers is small and partly balanced by the smoothness of the floor. The river bed remains steady and the height to which floods will rise is known. The bed of the river in the neighbourhood of the work can be manipulated in a surprising way so as to remove shoals or accretions. The front of head sluices can be kept clear. The system is also very useful in the case of canals that have to cross rivers on the level. The works themselves are not difficult to construct, and do not need to be particularly solid or to have very deep foundations, since the interference with the stream is small.

It would be expected that the important innovation in these shutters would be the rollers. This is not so. The rollers do not in practice revolve at all freely, often not at all, and the shutters do not work sweetly or easily. The truth is that the shutters are effective because of their great weight, because of the counterweight and because of the great power applied at slow speed. They do not really depend on diminution of friction. If really large, they have to be heavily counterweighted in order to rise without trouble. For lowering, the counterweight must be lifted, and then the great weight takes the shutter down. This means that the counterweight cannot be hung by a rope from a loose pulley, but must be hung by a chain from a chain-wheel or sprocket wheel. Even with the best Admiralty chain there is then some stretching, the links do not fit the teeth, and there is jerkiness and straining. The rollers are really only of service to give play and prevent jamming. In the opinion of the writer, Stoney's sluices, which really do depend on absence of

friction, are far superior, and Colonel Smart would probably now admit it. At the time he designed these shutters Stoney's patent was still running and Stoney's shutters were manufactured by a single firm in England, of which the inventor was works manager. They were in consequence exceedingly expensive. The patent has now expired and there is no inherent difficulty in making them, nor should they be more expensive than Smart's shutters. At the present moment a Stoney shutter is being made at the Public Works workshops at Madras, and is to be put up among a series of Smart's shutters at the Upper Anicut near Trichinopoly in order to test the merits and expense of the two. In the Tungabhadra project an immense number of large shutters is required, and the writer has entirely abandoned Smart's pattern and has obtained designs from Messrs. Ransomes and Rapier on the Stoney principle. Some of these are very large, *e.g.*, there is one installation of shutters 40 feet span by 30 feet high, and another of shutters 80 feet span by 9 feet high. These would, it is believed, be quite impracticable on Smart's system.

There is of course a considerable difference in cost between a weir, a regulator with Smart's shutters, and a regulator with Stoney's shutters. There is a proposal to build a regulator across the Penner river at a point where the width is about 1,700 feet. A weir similar to that at Bezwada on the Kistna would cost about three lakhs. A regulator with Smart's shutters 40 feet × 9 feet would cost over six lakhs. A regulator with Stoney's shutters 80 feet × 9 feet would cost about eight lakhs. In the latter case the English quotation for each shutter is Rs. 31,100, which might be good deal reduced. In the case of a weir the river on either side would have to be banked, and maintenance would be much more costly.

A weir, as we should now build it, would cost considerably more than three lakhs.
A. H. G.,—3-8-01.

PAPER No. 2.

Automatic waste weir gates of Bhatghar reservoir (Lake Whiting).

The sluice gates described in this paper are working on the waste weir of the storage reservoir, of the Nira Canal at Bhatghar, about 30 miles south of Poona. The reservoir, which, after the Periyar lake in Madras, is the largest in India, has a capacity of 5,313 million cubic feet. It has a water-spread of 3,584 acres at the full supply level. Including the waste weirs, the dam is 4,067 feet long, and its height above the foundations is 127 feet at its deepest part.

The catchment area above the dam is 128 square miles with a varying rainfall averaging 145 inches annually. The maximum estimated flood is 51,200 cubic feet per second equivalent to a run-off of $\frac{5}{8}$ inch per hour from the whole catchment.

The construction of the dam was commenced in 1881 but the design for the waste weir was not settled, and although the reservoir was in partial operation, the weir remained incomplete, till the year 1891.

In common practice the solid waste weir crest represents the full supply level of a reservoir. The height of flood over the weir crest depends on the quantity of water to be disposed of and the length of the weir.

Although the dam may be designed to hold up water to the level of top of flood waterway, the available storage is only what remains up to the solid crest after the subsidence of the floods. The capacity of the reservoir between the crest of weir and the maximum flood level is so much waste space for the purpose of storage. The problem before the engineers connected with this dam was to provide means for utilizing this waste space for increasing the storage, without raising the level of water above the height for which the dam section was designed.

The first engineer to attempt a solution of the problem was Mr. J. E. Whiting, M.I.C.E., who designed the Nira Canal project, and who had also a large share in the supervision of the construction of that work.

He designed a falling gate and it was proposed to accommodate 45 of these on the north waste weir which was built up to within 3 feet of sill of the proposed sluice openings. At this stage, an experimental gate was erected and tried on the main river weir at Vir, the head-works of the Nira Canal. It was self-opening, and was worked by a counterweight in a chamber of the solid masonry wall of the weir. A passage was left in the pier at the highest flood level for the water to flow into the counterweight chamber. An opening was left at the bottom of the counterweight chamber which was always slightly open. When the water which came in at the top inlet exceeded the quantity which passed off at the lower exit, the chamber gradually filled, the counterweight lost weight and the gate was laid flat on the floor of the sluice. It was expected that when the inflow into the counterweight chamber ceased and the chamber was emptied, the counterweight would regain its full weight and lift the gate into position. The gate opened automatically but caused great difficulty in closing. It would not lift into position until the water was lowered to about 2 feet above the weir crest. As this was considered a serious defect, Mr. Whiting's design was abandoned.

Mr. LeQuesne, the Executive Engineer of the Nira Canal at the time, suggested the adoption of a more sensitive gate invented by Mr. E. K. Reinold, then Executive Engineer of the Poona District (Roads and Buildings Branch). The latter officer worked out all the details of manufacture and erection in consultation with Mr. LeQuesne, and Government duly sanctioned an experiment with the new gate at Vir and appointed a Committee to enquire into and report on its merits. The Committee were very favourably impressed by the invention. In paragraphs 8 and 9 of their report, they said :—

"The Committee are unanimously of opinion that the automatic action of the gate, both as to opening and shutting, is practically perfect. They are

further of opinion that it is superior to every form of automatic gate they are acquainted with, for storing water in reservoirs to the depth of maximum flood level, or in other positions under similar conditions, because it is simple in design, completely under control, and so sensitive and certain in its action that the smallest loss of storage water is practically impossible.

As regards the leakage observed in the experimental gate, the Committee consider it of no consequence whatever, in the case of the Bhatghar and Gokak weirs. But supposing that in future gates of this design, it were impossible to reduce the amount of leakage, they would still unhesitatingly recommend this form of automatic gate being adopted in both the works above referred to, and they think it difficult to over-estimate the value and importance of the invention for similar purposes."

Mr. Reinold's gate works a sluice opening 10 feet wide by 8 feet high. It moves on 4 wheels or rollers, two each fixed to either side of the gate, and has the appearance of a truck or trolly moving on vertical grooved runners on the side frames of the gate, instead of on horizontal rails. In ordinary sluice gates, the gates slide on iron frames round the orifice. In Mr. Reinold's design they move on rollers, and the gate and frame do not come into contact except in the final position when the sluice opening is fully closed.

The power required to move the gate has therefore to overcome only rolling friction instead of sliding friction, and there is an enormous saving in the power required to open or close the shutters as compared with sliding shutters.

The gate is constructed of wrought iron plates and angles as shown in detail on Plate 9. The wheels are of cast iron with the outer rim turned smooth and true and the bearing bushed with brass bored smooth and accurate to fit the axle easily. The axle is of mild steel. When the gates hang freely, all four wheels bear evenly on the grooved runners of the frame. The frame is of cast iron and consists of two vertical side pieces bolted together in the centre at the level of sluice sill by a horizontal piece. The distance between the centres of the grooves in the two side runners corresponds accurately to the gauge of the wheels on the gate.

A special feature of this gate is the taper joint between the gate and the frame, intended to stop leakage when the gate is closed. The joint is formed by strips of cast iron which are faced with $\frac{1}{4}''$ brass and bolted to the bottom and sides of the gate. Corresponding plates of cast iron similarly faced with brass are bolted to the bottom and sides of the cast iron frame fixed on the three sides of the sluice opening. The plane of the surface of the three strips on the gate and that of the strips on the frame are sloping or tapered in opposite directions as shown on Plate 9. The gate moves vertically on its rollers from below the sill to the highest position, in which the gate closes the opening, and the strips of the gate and frame come in contact and form a water-tight joint.

There are two set screws fixed in brackets against the bottom of which the top corners of the gates abut when fully closed. By lowering the screws the gate is detached from the frame and kept at a small distance away from the strips on the frame. The extreme distance to which the screws have to be lowered is not more than 1 inch. When the screws are lowered, the top of the gate rests against them and prevents the forming of an absolutely water-tight joint. The object is to prevent the gates becoming jammed against the frame in times of flood. The loose bearings permit a little leakage, but it is of no consequence since the river which feeds the reservoir is in flow for 2 or 3 months after the close of the monsoon. After all fear of storms has ceased, the set screws are moved back and the gates allowed to come in contact with the frame, thereby forming a water-tight joint. It may be added that the bottom and side strips or plates referred to above have slot holes in them to allow of their being moved slightly towards or away from the gate according as it is intended that the gates should be water-tight or easy to move.

The gate is attached at the two top corners by chains taken over pulleys above it and attached to a single counter-balance weight working in a chamber in the body of the weir wall at the back of the sluices. One chain is taken

diagonally across sluice opening and the other along a pier, so that both chains enter a well in the masonry at the lower end of the pier. Water is kept out of the opening by vertical wrought iron cover plates fixed on either side. The chains descend vertically into the manhole and are attached to the counterweight. The gate including the concrete addition weighs 6,800 lbs., and the counterweight, including the sand ballast, 7,300 lbs.

As stated already, the counterweight chambers had been constructed according to Mr. Whiting's original design and the waste wier brought up to within 3 feet of the sill level of the sluices, when Mr. Reinold's patent gate was adopted. As the same chambers and outlet channel were utilized, the working of the gates is similar in action to Mr. Whiting's experimental falling gate described above. When open, Mr. Whiting's gates fell flat on the floor of the sluice; Mr. Reinold's gates descend vertically to a position below the sill of sluice opening.

At the extreme north end of the sluices is a tower which admits water into the counterweight chambers by means of two sluices, one a pipe sluice of 12" diameter fixed at 3 feet below the sluice sill, and another a gate 24" × 15" fixed 6' above sill of sluices. At the bottom of all counterweight chambers runs a longitudinal drain which is 15" wide with a depth of 6" at the north end, increasing gradually to about 18" at the south end. This duct is joined at both ends to cast iron pipes controlled by sluice valves. The sluice valves in the valve tower are intended for working the sluices when the water is below the flood level. The sluice valve at the lower end is kept always partially open.

When a flood occurs, water rises to the level of, and enters, the inlet openings in the piers. The counterweight chambers are filled and the gates go down with a slow motion below the sluice opening. When the flood level again falls below the level of the top openings, the chambers are gradually emptied by the bottom masonry drain referred to above. The counterweights descend as the chambers empty, and the gates are lifted and made to close the sluice openings.

The waste weir openings have a clear width between piers of 10 feet. The openings are arched over and provided with a road on top in continuation of the roadway on the main dam. Rails are laid on the roadway for a crane, for handling the gates for examination or repairs.

Openings are left in all chambers to provide an overflow on the downstream side.

The erection of the gates was completed in 1895. Some trouble is experienced when water falls below the sill-level of gates, and the gates move about violently and ram against the side frames in high winds and storms. This difficulty is not experienced however when the lake is full and the gates have to work automatically. To prevent damage, when hanging free in air, the gates are tied to the frames by temporary clamps which are removed as soon as the level of water rises to a height of about 3' above sill. The officials in charge are required to be careful to remove the clamps in times of flood.

Experiments have been made to find out the extent of leakage when the bearings are loose. The leakage through each gate, at one of the experiments, amounted to ·47 cusec per gate or about 21 cusecs for all 45 gates on the north waste weir. When the bearings were made water-tight, there was practically no leakage. At one of the trials, the leakage through the 45 gates did not exceed one cusec.

Fifteen out of 45 gates are thoroughly cleaned and painted every year, so that every gate gets its turn once in 3 years. All the counterweights, chains and pulleys are examined before the commencement of the monsoon. All bolts and nuts which can be detached are cleaned, oiled and restored. The gates are covered with a coating of Dr. Angus Smith's solution. The brass faces of the side strips of the gates and frames are coated with tallow. Before the flood rises to the full height, the gates are tested and worked at least once a week. The side strips of the frame are set back during the monsoon and the top screws on the brackets of the side frames lowered in order to keep the

bearings loose and to ensure smooth working. At the close of the monsoon the strips on the frames are pushed forward and the top screws drawn back to make the gate come in contact with the frame and prevent waste of water.

A care-taker and four watchmen are employed to attend to the details. For a fortnight or so after the close of the monsoon, the men have to be watchful in order to see that in the event of sudden storms without warning the gates are rapidly adjusted to the position they should be in a monsoon. No difficulty has however been experienced in this respect till now, and during the five years they have been under the author's observation, their working left nothing to be desired.

Besides the 45 gates on the north waste weir, there are 36 sluice openings on the south weir which are closed by roller gates of Mr. Reinold's pattern. These gates were not made to work automatically because the fall on downstream side was considered insufficient for the application of the automatic principle. These 36 gates are worked by the aid of a crane by the care-taker and watchmen mentioned above. The water level in the lake is gradually raised during the monsoon and the full height is secured only towards the close. The men in charge keep open the south waste weir non-automatic gates, whenever the lake is overflowing; the automatic gates come into action when the floods are too large to be dealt with by the non-automatic gates.

The gates were manufactured by Messrs. Geo. Gahagan & Co., of Bombay. The cost of the ironwork of the 45 automatic gates (excluding erection for which reliable figures are not available) amounted to Rs. 89,078.

The storage of the Bhatghar Reservoir below the crest of solid weir is 3,900 million cubic feet and that above the weir crest, impounded by the gates, is 1,413 million cubic feet. The supply secured by the gates represents therefore an addition of about 36 per cent. to the available storage. This substantial addition is a testimony to the value and importance of gates working on the automatic principle.

PAPER No. 3.

Automatic waste weir gates of Lake Fife, near Poona.

The fair-weather supply of the Mutha Canals is stored in Lake Fife, which is formed by a dam across the Mutha River at Khadakwasla about 10 miles south-west of Poona. The catchment area of the lake is 196 square miles, and the estimated maximum flood 62,000 cubic feet per second, equivalent to a run-off of ½ inch per hour from the whole catchment.

The completion of the waste weir of this dam has been a subject of discussion ever since its construction was commenced. By the year 1880, a portion of the weir had been raised to the designed full supply level and the remainder brought up to within one foot of that level. Temporary banks of fascines and earth were built to store an extra supply of about 18 inches over the new crest but the banks were not reliable. Time after time, attempts were made to store water by temporary banks in this way but the arrangements proved defective, and whenever a flood occurred after the bank had been put up, the latter was damaged or cut away and the extra storage lost, often beyond hope of future replenishment in that season. In 1884, arrangements were made for storing a depth of 4 feet of water above the weir crest by means of a movable weir of teak planks fixed in iron standards. The standards and planks were placed in position at the close of the monsoon and removed after the extra supply stored had been consumed. This was an improvement on the earthen bank, but was attended, though in a smaller degree, with the same risks in the event of a sudden flood. The canal irrigation sluices were tried for a time and subsequently 12 regulating sluices, each 10 feet × 6½ feet, were added in order to be able to pass off all small floods after the planks had been fixed and the reservoir filled.

In July 1896, with an open weir, a flood rose 18 inches higher than the limit of safety originally laid down. This led to a reduction of the weir crest, which till then was in two levels, to one uniform crest 3 inches above the lower level. Floods of considerable intensity occurred occasionally, towards the close of the monsoon also. In September 1900, a heavy flood occurred when the planks of the temporary weir were up and had impounded 3½ feet of water above the permanent crest. The flood overtopped the movable weir and rose 3·65 feet above the top plank and 1·40 feet above the safe maximum flood level.

When the weir is obstructed by a temporary barrier of standards and planks before the close of the monsoon, the floods may and do rise to a dangerous height. The necessity of removing this element of risk to the dam was all the greater on account of the situation of the city of Poona below the lake. With a favourable monsoon and floods of small intensity at its close, the arrangements then existing worked tolerably well. Under other conditions, there was risk on the one hand of loss of storage by too much caution, and, on the other, of a rise of water level above the limit of safety. A temporary weir of boards, nearly a quarter mile long, is not easy to manipulate under the best of conditions. When a flood overtopped it, the men employed lost control of the arrangement and looked on helplessly till the flood had worked its will.

It was therefore evident that either the movable weir should be abandoned and the large amount of extra storage (equivalent to about 150 million cubic feet per foot of height of temporary weir sacrificed with it, or a more efficient movable weir should be provided which could be automatic, both as to opening and closing, or which could be more easily worked.

The automatic gates of the Bhatghar pattern where each gate has a separate counterweight were considered unsuitable for the conditions of this weir on account of insufficiency of the fall available on the downstream side over the greater portion of the weir. Another difficulty was that in order to construct counterweight chambers in the masonry which the Bhatghar arrangement required, it was necessary either to build a new weir downstream of the old

weir, or to dismantle and rebuild the greater portion of the latter. The dismantling would have involved the sacrifice of some portion of storage and a corresponding loss of irrigation and revenue for a time.

To meet the special conditions of this weir, the new appliance described in this paper was designed and patented by the author. The design was sanctioned as an experimental measure for a set of 8 gates in 1901. Mr. John Tate, M.I.C.E., the late Chief Engineer of the Bombay Government, to whom the project owes its inception, appointed a Committee to report on the gates. Mr. C. N. Clifton, Superintending Engineer, Central Division, who has been closely associated with the design in all its stages was President of the Committee. Mr. Hill, C.I.E., was also specially requested to examine and submit a report on the gates. The Committee reported that " the design was eminently suitable to the special conditions at Lake Fife ". The first set of 8 gates, which was subsequently completed and tested in the floods of 1901, worked very satisfactorily.

The gates are in sets of eight. Each gate covers a clear opening 10′ wide by 8′ high, so that one set of eight gates worked by one counterweight occupies a length of about 110 feet. Eleven sets of gates have been constructed over a length of about 1,208 feet of weir. Set No. 1 was erected, as already stated in 1901; sets Nos. 2 to 11 have been completed since. Eighty gates or 10 sets have been worked automatically for the full head in the lake. The remaining set of 8 gates which is of the all rising pattern has been worked under a head of 6 feet. The adjustment of this set for the full head is in progress.

The gates or shutters used are those invented and patented by Mr. E. K. Reinold. The gates move on wheels or rollers against the sluice frame whereby all surface sliding friction is avoided and the frictional effect of the water pressure on the gates is much reduced, being converted into axle or rolling friction.

The surfaces of bearings of the sluice frame and gate are tapered in opposite directions so that when the gate is closed and the surfaces come in contact the joint becomes watertight. In the designs, hereinafter described, a number of sluice gates (in this particular case, eight) are actuated automatically or otherwise by means of one counterweight float. Instead of each gate being worked directly by a counterweight, as in the Bhatghar arrangement, either the gates are balanced one against the other, the resultant weight only going to the counterweight (Design No. 1) ; or an intermediate balance weight is introduced, whereby the gates are partially or wholly balanced and the weight and size of the main counterweight reduced (Design No. 2).

Design No 1 is illustrated in Plate 10. The gates, on the weir face, work in pairs. They are suspended from pulleys, and the members of each pair partially balance one another. The sluices open when the heavier gate of the pair is released and allowed to fall. When open, the heavier gate of the pair is lowered below the sluice opening and the light gate is taken clear above the flood water level (Plate 10).

The falling gate is loaded and is the heavier of the two and weighs from 100 to 120 cwts., according to the specific gravity of the material used in loading. The rising gate weighs about 44 cwts. Each pair of gates is closed by hauling up the heavier one by means of a chain which is attached at its other end to a line of chains or wire ropes, moving on pulleys and rollers, and actuated by a counterweight working in a cistern below the waste weir.

The counterweight for gates of combined pattern is formed of a hollow water-tight cylinder with a cubic capacity or displacement of 760 to 860 cubic feet. The diameter and height of the cylinders vary with the depth of overfall and the nature of the site. The counterweight is filled with sand to the requisite extent. In the design adopted at Lake Fife, one counterweight is made to work eight gates as stated already.

The action of the counterweight will be understood by a reference to Plate 10. An inlet channel made in the pier with its mouth at P (Fig. 6) leads from the reservoir into the counterweight cistern. The sill at P is about 6 inches below the level of the full supply. An outlet pipe is laid from the bottom of the cistern and taken to the nearest point beyond the waste weir.

channel where it can discharge freely into air at all times and above the level of tail water in the waste channel. The outlet pipe from each cistern is throttled by a sluice and so regulated that the flow from the pipe is always less than the maximum flow into the cistern through the inlet. When water rises to the full supply level in the reservoir, it begins to flow into the cistern through the masonry inlet, and gradually immerses the counterweight. The latter floats, the tension on the chains attached to the falling gates is slackened and the gates thus released go down by their own weight. When the water in the reservoir falls below the inlet level the supply to the cistern is cut off and the cistern is gradually emptied by the outlet pipe already described. The counterweight gradually regains its weight and, as it falls, the heavier one of each pair of gates is pulled up and the lighter one falls by its own weight, and the sluices are closed.

The gates can be worked whenever the level of water in the reservoir is below the highest flood level, by means of a pipe sluice fixed about 3′ below the weir crest. When there is no water in the reservoir above the inlet pipe, they may also be worked, for purposes of testing, by filling the cistern with a hand pump.

The counterweight cistern is placed behind the middle pier on the downstream side of weir. It slightly deflects the current from the nearest sluices but otherwise has no effect on their discharge. It is found by actual observation, since the sluices have been working, that with $7\frac{1}{2}'$ of water on the upstream end of sluice pier, the water level about 10 feet below at the downstream end falls to nearly 4′ at the counterweight pier, showing that the cistern has practically no effect on the discharging capacity of the sluice openings on either side. The pier and cistern have also other uses. The former acts as an abutment pier for the row of arches and the latter as a buttress to the weir.

The rate of flow of water into the cisterns can be regulated and the cisterns filled and emptied slowly. The counterweights rise and fall with a slow steady motion and are not subject to sudden movements or jerks. Valve chambers are provided in front of sets 1, 6 and 10, with sluices for admitting water into the counterweight chambers at various levels. By means of these valve chambers, water can be maintained in the lake at any desired height above the sill of sluices. Whenever water rose above the regulated level, it would flow into the cisterns and the whole series of gates would open automatically.

In No. 2 arrangement, Plate 11, the gates are all made to rise in opening They work exactly like the rising gates in Design No. 1, but all the 8 gates in this case are attached to a heavy balance weight suspended by chains in a chamber built in the centre pier.

The gates are marked B B on Fig. A1, Plate 11, and the balance weight by the letter F.

The balance weight is attached in one direction to the 8 gates and in the other to the counterweight float. It is formed of a cast iron tank resting on steel rails and filled with blocks of lead to economise space. The counterweight float together with the weight of the gates B B is made sufficiently heavy to overcome the balance weight, so that when there is no water in the cistern, the counterweight is in the lowest position, the balance weight is up and the gates go down or close. When water rises to the level of full supply, it enters the counterweight cistern by the inlet opening shown in Fig. A6, Plate 11, the counterweight W loses weight by displacement and the balance weight regains its weight and, falling, pulls up or opens the gates. The water may be admitted into the counterweight cistern at various heights of flood and the gates made to work, as before, by means of a valve chamber, or by the lower inlet pipe as in No. 1 pattern.

The waste water pipes of the eleven cisterns are joined to one pipe which ends at the point beyond the wing wall of the waste weir and discharges into the deep natural valley of the river at that end.

An opening for overflow is provided in every counterweight chamber to prevent it from filling too full whenever the waste water pipe is temporarily surcharged.

The sluice openings are crossed by arches on the top of which are fixed in suitable masonry passages the several pulley brackets intended for carrying the chains from the gates to the counterweights. Rails are laid over the roadway on top to carry a 10-ton crane for use in handling the gates.

The design adapts itself to weirs with little or no overfall and almost to any site. The gates can be fixed on the top of a high masonry dam by constructing one counterweight for eight, 12 or 16 gates on the rear slope of the dam.

In low weirs, or where there is no fall at all, the waste water pipe from the counterweight chamber may be taken on to the river bank, and thence continued along the bank till the fall required is gained.

Where the country is flat, the gates of the rising pattern can be constructed across streams or rivers and the waste water pipe led into a sump well on the river bank. The well may be emptied by a hand or bullock pump whenever it is required to close the gates and to head up water. They will always open automatically in flood time, which is the most important consideration in movable weirs.

In the combined pattern design, the falling gates only require to go below the sill; all other ironwork will be practically above water, always accessible and under perfect control.

Similarly all gates, counterweights and balance weights of the rising pattern are at all times accessible for examination or repairs.

In the combined falling and rising pattern the heavy gates are loaded with lead or cast iron, and partly with cement concrete with a view to save expense. The gate itself with the top frame weighs nearly 60 cwts. and including the weights, 100 to 120 cwts. according to the specific gravity of the material used in loading. The light gates weigh 44 cwts. each, and the counterweight float of each set weighs about 400 cwts., when adjusted for working under a full head. As the work is yet new, it is possible that a reduction of the weights may be effected after further adjustment and experience in working the gates.

The pull on the gates was observed by means of a Denison's scale for the various heights of opening under the full head of 8 feet above sill. The results of the observation on one heavy and one light gate were as under :—

Height of opening of gate.	Pull on heavy gate.	Pull on light gate.
Feet.	Cwts.	Cwts.
2	88½	59¼
4	83¾	51¾
6	84¼	47¼
8	73¼	44¼

The weight of this particular heavy gate in air was 115¼ cwts. and of the light gate about 44.

The gates used are made of the best wrought iron plates and angles. To the vertical sides and the lower horizontal plates are bolted strips of cast iron, faced with muntz metal. These strips form the faces in front (downstream side) of the gate corresponding to similar faces on the frame and are intended to form a water-tight joint between the gate and the frame. Each strip is planed true from end to end and faced smooth so that when the three are bolted to the gate they are accurately in one plane.

The water pressure on the gate is taken, as already described by 4 wheels which work in the grooves provided for them on the frame. The wheels are of cast iron with the outer rim turned smooth and true, and the bearing bushed with gun metal bored smooth and accurate to fit the axle easily. Each gate when hanging free has the plane of the 4 wheels accurately in a vertical position and the grooved runners of the frames are fixed vertically, and all the four wheels bear evenly when the gate is hung.

The frame consists of two vertical side pieces bolted together in the centre by a horizontal piece. The three adjustable faced strips bolted on the frame correspond accurately to the muntz metal faced strips on the gate. Screw stops are fixed to the top of the light gates, and serve to keep the gates detached from its frame during flood time in order to allow the gates to work freely.

The pulleys, both frame and sheave, and all brackets are of cast iron, and the axle pin is of steel of the best quality turned smooth and true.

The counterweights are wrought iron cylinders or tanks closed on all sides and made perfectly water-tight at site.

Shackles, adjusting links and screw couplings, are provided on the chains for connecting, tightening or loosening them as required.

As a practically permanent area of sugarcane was dependent on the storage in the lake, it was necessary to dismantle the weir and rebuild the works in one season, in order to avoid loss of storage. This caused much difficulty and a substantial addition to the expenditure, on the erection of the gates, which otherwise would not have been necessary. The first set was completed towards the middle of the monsoon of 1901. The remaining works were sanctioned in September 1902. By the month of August following, the work was sufficiently advanced to enable $7\frac{1}{2}$ feet of water to be retained in the lake by temporary arrangements. All the remaining gates have been fixed in position since and although the adjustment is still going on, all but the 11th set are working automatically with a full lake.

The gates were manufactured by Messrs. Geo. Gahagan & Co., of Bombay, who took great interest in the details of the design.

The cost of the ironwork of the 10 sets, 2 to 11, or 80 gates, amounted to Rs. 2,54,876.

The storage impounded by the sluices above the sill is 1,244 million cubic feet, or 46 per cent. of the total available storage of the lake. The works cost of the gates, including the masonry but excluding the first experimental set, amounts to about Rs. 4,48,530, giving a rate of Rs. 361 per million cubic feet stored. The total cost, including establishment, tools and plant and indirect charges, will be about Rs. 5,21,010, which gives a rate of Rs. 419 per million cubic feet. If the storage obtained formerly by the standards and boards be deducted, the net increase will amount to about 525 million cubic feet, giving a rate of Rs. 992 per million cubic feet. The great gain is a fixed, instead of a varying, storage, and the removal of danger to the dam and to the city of Poona situated below it.

The calculated discharge through each sluice opening is 751 cubic feet per second, and the 88 new sluices are estimated to dispose of 66,097 cubic feet per second. The total discharging capacity of the weir, including the old and new openings at either end, is estimated at 74,877 cubic feet per second, representing a run-off of 3/5 inch on the catchment.

Each set of 8 gates takes about 7 minutes to open and 8 minutes to close when the valves are worked by hand. The sluices of all eleven sets can be opened or closed by one care-taker in about 20 minutes. In this interval a flood of 8 feet depth, and nearly a quarter mile wide, is let loose or controlled by the sluices.

PAPER No. 4.

On Falling shutters on the Paricha Weir.

These gates were erected on the weir crest in 1900-1901 with the object of adding to the storage capacity of the reservoir without increasing the maximum flood levels.

The considerable length of the weir, nearly ·75 mile, and the sudden and intense habits of the floods, led to the adoption of an automatic falling gate, in order to avoid risks to the works and establishment, which it was thought would attach to the use and working of non-automatic gates, especially at night time.

The principle of the action is illustrated in the diagram Fig. 1.

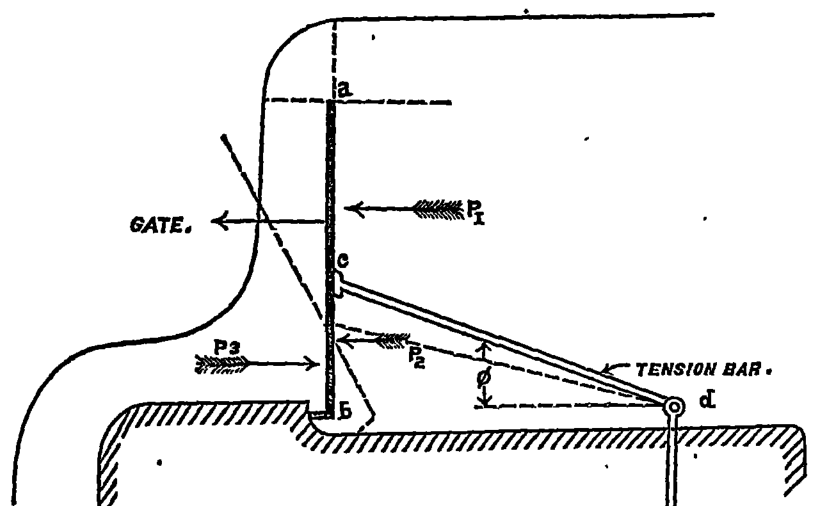


FIG. 1.

The gate a c b is held up against the water pressure by tension bars c d; it is in equilibrium so long as the moment of the pressure P_1 round the point c, on the portion of the gate above c, is not greater than the sum of the moments about c of the pressure P_2, and of the resistance to sliding of the shoe at b. When it exceeds this, the gate upsets automatically in the direction indicated by the dotted lines.

The general formula for determining the position of the point c which will give equilibrium for any given depth of water over the gate is—

$$P_1 \times \tfrac{1}{3}\, ec = P_2 \times (\tfrac{2}{3}\, eb - ec) + C_f\, [\, (P_1 + P_2) \tan\phi + W\,]$$

where P_1 and P_2 = the hydrostatic pressures above and below c; W = the weight of gate; and C_f = a coefficient of friction for the materials in sliding contact at b. This formula does not take account of the back pressure due to the water falling at the back of the gate P_3, which is variable and not exactly calculable; hence the position of c, determined by calculation, requires check by practical experiment.

In order to avoid undue shock and too sudden increase to the flood discharge over the weir, the gates were designed to fall in three flights as follows:—

Flight.	Number of Gates.	Designed to fall with depth over top of—	Calculated height of C above b as finally adopted.
A	111	2·0	2′—0¼″
B	100	3·0	2′—1½″
C	100	4·0	2′—2⅜″

The dimensions in the last column were those finally adopted after practical experiment, the gates being all 6·0 feet high.

The plan appended illustrates the gate in detail. Plates. 3 and 14.

It may be noted that since erection, nearly all the gates have buckled slightly along their top edge. I think the method of bracing is defective, and both it and the top angle stiffener are scarcely heavy enough. The buckling is not sufficient to impair their efficiency.

It is found that the gates can be raised by hand more rapidly than by the portable shears originally provided for the purpose. After the floods of 1903 all the gates were erected in 1½ hours.

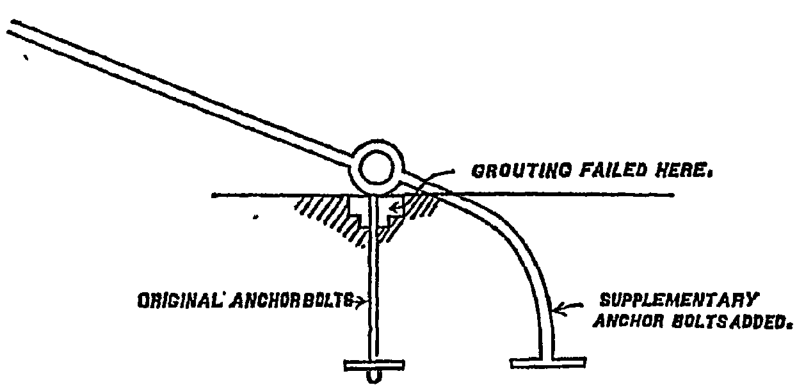
Lowering by hand is more tedious and will be noted on later.

Under standing orders the gates are lowered by hand when floods are imminent, and hitherto this has always been done in ample time; hence their automatic action has only been tested on two occasions, in 1901 and 1903, when some of the gates were left standing intentionally.

In the 1901 test, the few gates of the A series left standing all upset at the approximate depth for which they were designed: the B and C series were not tested.

As a result of this test it appeared that the anchor bolts were not strong enough, or rather that, owing to the failure of some of the grouting material, they were subjected to strains for which they were not designed; as 2 or 3 were unsupported, they were subjected to bending moments in excess of expectation, bent unequally, and the gates and tension bars were consequently distorted.

This was met by introducing supplementary bolts as sketched.



This defect indicates that it would be better to fix such bolts in the direction of the pull, instead of vertically as is generally done. But I see no advantage in the curved bolt over a straight one, which is easier to fix.

In the 1901 and 1902 floods one or two of the gates and tension bars, on sections of the weir which are liable to have stones washed over them, were found damaged by stones getting underneath them. In one case a tension bar was snapped in two. This has been met by the provision of clip bolts to hold down the gates which are liable to this action. These, however, can only be applied when the gates are lowered by hand; when allowed to fall automatically, it is obvious that the clips cannot be applied, and that under these circumstances the gates will always be liable to damage from this cause.

This liability is now being minimized by the partial removal of the islands obstructing the weir, and the precaution is also adopted of keeping the islands upstream of the weir free from loose stones.

But the experience goes to show that this type of gate would not be suitable for low weirs in boulder bed rivers.

In October 1903 the gates were subjected to a more general test; all except 75 of the A series were left standing during the October flood, which reached a level of 2·1′ over the top of the gates.

Their action is detailed in the following statement:—

Series.	Depth on top at which designed to fall.	Total number of gates.	Number lowered by hand.	Number upset by flood 2·1′ over top.	Number which remained standing.
A	2′	111	75	21	15
B	3′	100	*Nil.*	68	32
C	4′	100	*Nil.*	15	85

The few A gates which remained standing were all masked by adjacent obstructions.

The B gates which remained standing were also nearly all partially masked.

Those that fell were on a clear section of the weir. The falling of these shutters may be due to the fact, observed and recorded by Mr. Laurie in the 1902 flood, that the water in midstream was appreciably higher than at the sides. It is conceivable that they had fully 3·0′ depth over them before they fell, although the flank gauge recorded only 2·1′.

The C gates did not fall in series as did the A and B, but singly or in couples at irregular intervals, the intermediate gates remaining standing.

The explanation suggested is that the water falling over them set up vibratory action, which gradually disturbed equilibrium, the frictional resistance of the shoe and other moving parts of the gates that fell being less than in the others. If this explanation is correct, it indicates that these gates can be relied on to preserve their equilibrium only within certain time limits, and that long continued flood action over them gradually induces them to fall with less than the designed depth. This is not necessarily an objection to their use, and certainly not at Paricha.

During this test none of the gates, tension bars, or anchor bolts were damaged in the slightest degree; the gates behaved well.

Reference has already been made to the comparatively slow rate at which the gates can be lowered.

The 75 "A" gates lowered before the 1903 flood occupied 17 men three hours. The complete set would at this rate have taken the men 12·5 hours nearly.

The flood was rising only gradually when the lowering was stopped, and the gauge records indicate that all could have been safely lowered in advance of the flood, but the next day they could not have been safely lowered by hand as the flood rose some 5 or 6 feet in as many hours.

In the 1902 flood also the rate of flood rise was gradual on the first day, and all gates were safely lowered by hand in plenty of time, but on the second day the flood rose 7·5′ in six hours and 8·2′ in twelve hours, while the maximum rate was 3·5′ in one hour.

Thus, while it appears that it is as a rule possible to lower all the gates safely by hand, it is quite conceivable that with a full reservoir a flood might come down too rapidly to admit of this being done without risk, and the provision of automatic gates at Paricha was doubtless prudent.

On the other hand, there are several patterns of non-automatic gates which could be lowered in one-fourth the time it takes to manipulate the Paricha type; and certainly if the weir were somewhat shorter, I think there would be no necessity for automatic gates at Paricha.

In cases where conditions admit of their use safely, I should prefer non-automatic gates with back struts to the Paricha pattern, as being less liable to damage and probably cheaper.

In conclusion, I may note that as a result of several practical tests "Siderosthen" has been selected as the most economical and efficient paint for use on these gates and fittings.

PAPER No. 5.

On the Khanki Weir Shutters.

The gates and its setting :—These shutters or drop gates are 6 feet high and 3 feet wide and are hinged at their lower ends to the crest of the weir. The frame is built of two side angle-iron pieces, another at the top, and a channel iron at the bottom. It is stiffened by two cross-pieces of angle-iron, and the face is covered with a $\frac{3}{16}''$ steel sheeting. The drawing on Plate No. 14 shows the final approved design with all improvements that have been carried out. The double hinge blocks are placed between two adjacent gates, and are held down each by 2 bolts, 2′ 9″ long. The catching arrangement consists of 1″ diameter tie-rods, hinged at one end to $1\frac{1}{4}''$ diameter anchor bolts at a point 3′ upstream of the shutters, and buried 3 feet 4 inches in masonry, while the other end is provided with a steel tie-pin for attaching to the trigger arrangement. When the gate is prostrate the tie-rod lies in a guide groove, ready to slide up as the gate is raised. A slot in the face sheeting allows the tie-pin to pass through the gate to the back, where it can be caught by the trigger. The trigger is provided with a pivot bolt at its lower end, beneath the point of contact with the tie-pin, and is grooved or forked at its upper end. It is guided by an annular plate, and when adjusted with the tie-pin, it is kept in position by the pressure on the gate. The groove is intended to carry the rope of the automatic dropping arrangement described further on. For lifting the shutters by mechanical means, an angular sling has been provided on the up-stream face, at a point $1\frac{1}{2}$ feet from the top. The vertical sides are finished off with teakwood fillets, which permit of a close tight joint. The weight of one shutter is about 500 lbs. The cost of the shutters, including all improvements and incidental charges, was about Rs. 1,60,000, or Rs. 40 per foot run. The annual cost of clearing, tarring, repairing and keeping them in good working order runs to about Rs. 1,000, or Re. 0-4 0 per foot run. This does not include cost of establishment for working. The shutters have been designed to be strong enough to resist the pressure of a six-foot depth of wate in front, and also bear being lifted from the floor with the same depth of water over them. The power required to raise the shutters under the circumstances is estimated to be between 2 and $2\frac{1}{2}$ tons. As a rule, however, the head of pressure has been hitherto limited to 5·0 feet, as it is dangerous to allow the water to rise higher before beginning to drop the shutters, for fear of the water passing over them and rendering it impossible to get at and lower them. There has never been any shortage to canal-supply in consequence of this limitation. The height of shutters was, of course, determined by the weir crest level and the maximum supply level in the canal. In this connection the following levels of the Khanki Headworks are of interest :—

R. L. of weir crest.	left flank	722·03
	right flank	723·05
R. L. of top of shutters.	left flank	728·23
	right flank	729·05
R. L. of undersluice sill		715·00
R. L. of canal regulator sill		717·00
R. L. of canal bed at head		714·30
R. L. of maximum water supply in canal		725·30

This provides a margin of 3·0 feet, and it is seldom necessary to head up the river supply above R. L. 726·50 to get all that is required for the canal supply. This means a head of about 4·0 feet against the left flank shutters.

Manipulation of the gates :—There were originally 1,296 shutters on the Khanki Weir but there are now 1,288, owing to structural alterations which reduced the length of the weir.

For raising the shutters there is a lifting crane, with a projecting jib, of 3-ton power, but this crane, running along the crest of the weir behind the shutters on a railway track, though very powerful, is difficult to work and slow

in action. Silt deposits on the track interfere with the rapid movement of the crane along the weir crest, and the process of lifting is slow, for what is gained in power is lost in speed. It takes eight days, and more, for one crane to complete the job of lifting all the shutters. There has been no temptation to provide more than one crane, and even that one is seldom used. From time to time more or less successful attempts have been made to raise the shutters by means of a crab winch placed in a boat moored above the weir. Failures in this respect have been chiefly due to the fact that the subject has not attracted sufficient attention, owing to the much simpler method of lifting by hand having found favour. It is the usual practice at Khanki to lift by hand, and for depths of $2\frac{1}{2}$ to 3 feet over weir crest, three men find it quite easy to lift a shutter. The time occupied in doing this varies with the head of pressure, and amounts to three or four minutes for each shutter. At Khanki owing to the presence of islands upstream of the weir, and to cross currents, heavy silt deposits occur in parts of the weir and over the shutters left prostrate for some time. It is then only possible to lift them where the silt has been washed out by forcing a stream over it. This is always the cause of considerable delay in lifting the shutters, whatever the means employed to do so. The last few shutters in each of the eight bays into which the weir is divided, want more than three men to lift each shutter. The jib crane and even the boat and crab frequently come in useful then. According to present arrangements there are sometimes delays in getting the shutter up, but generally speaking, they have given great satisfaction. As an example of what can be done by manual labour, the conditions of the autumn of 1903 may be cited There were 4 feet of water over the weir right flank, when the operation of raising the shutters by hand was commenced in September, and over 5 feet when all the shutters had been raised. The conditions at the left flank were similar. The river supply fell at the rate of about 400 to 500 cusecs daily. For one week the canal supply was short every morning by about 200 cusecs of maximum capacity, which was made up in a few hours with day-light.

Dropping the shutters is a much quicker operation, and occupies less than one-third the time taken in lifting them. The triggers sometimes jam, and the tie-rod pins are liable to stick from the pressure of silt above, after the tamped shutters have been up many months together. The Chenab River floods rise slowly, and there is always plenty of time to lower the shutters as the flood increases. The operation of lowering can always be carried on faster than the rising flood. As a further precaution there is an advanced river gauge about 50 miles up the river, whence telegraphic warnings of a rising flood are received in ample time to make suitable arrangements at Khanki. In the original design provision was made for automatically dropping the shutters by means of a 1″ girth wire rope with a metal button at one end, lying in the trigger fork and winding on to a crab at the end of each bay. The necessity for using this arrangement has not presented itself so far.

The shutters are very seldom injured by falling on the hard floor of the weir crest. After the first one has been dropped, there is usually a cushion of water, carried by the out flowing stream, for the next shutter to drop on to.

In the winter, it is very easy to tamp the shutters with rags and old gunny bags. This takes about three or four days. To render the weir absolutely water-tight, it is necessary to throw up a sand bank in advance of the shutters. This takes from one week to ten days at Khanki, as the sand has to be carried in boats, usually from some distance. This bund is also necessary for an examination of the condition, and for the repairs to the weir surface and to the shutters.

The staff regularly employed consists of two jamadars and twenty beldars, all able-bodied men and good swimmers. During the winter it is occasionally necessary to call in casual labour to help the regular staff to rapidly complete the operation of raising the shutters, and tamping them again after a freshet.

Improvements and Defects:—In the original design the guide groove for the tie-rod was straight and narrow. This confined the tie-rod to the groove at all times. In the first year's working it was found that silt deposited above the gates, and even wood or stones lodged where the tie-rods were to come down

with the gates, and when the gates were finally dropped, some tie-rods got bent over the obstacles, or the groove flanges were damaged. This was remedied by splaying the lower ends of the guide grooves, in order to release the tie-rod where an obstruction was met. After the gate has been dropped the obstruction soon gets washed out, and the tie-rod then reposes in its proper position in the guide groove. This alteration has quite overcome the difficulty experienced, but on the other hand it has given rise to another difficulty which occasionally presents itself. Owing to the width of splay given, the guide groove is liable to intercept and collect small pieces of stone as they roll over the weir when the gates are down, and on the gates being again raised, the tie-pin forces the stones up into the slot to block its own passage, and then there is some difficulty in fixing the tie-pin. This defect could be almost wholly removed by making the sides of the groove parallel after a short distance of splay in accordance with sketch No. 3, Plate No. 14.

It has always been a consideration that, probably, conditions may arise at Khanki when reliance would have to be placed on the automatic Rope and Button method of dropping the shutters. In the original design the rope rested in the open groove or fork of the trigger, and it was soon found by experiment that the rope occasionally jumped out of the trigger fork of the near gate, when the strain in the rope was suddenly relaxed by one of the gates dropping. This defect was remedied by adding a swinging hinge or guard plate over the fork. The improvement will be found useful when the time comes to make use of the automatic arrangement.

There is one other slight defect which has not been remedied, and which it is hardly worth remedying now at Khanki, as it is not of pressing necessity. The top of the anchor bolt to which the tie-rod is hinged, projects above the weir crest and is liable, occasionally, to get bent and broken by stones rolling over the weir. The portions subject to injury are 3-inch wide and 3 feet apart, hence the chance of being damaged is not very great. The stones that roll over the weir come from the loose protection pitching upstream. The first remedy is not to use light material upstream of the weir that will wash over it in flood time. Countersinking the bolt head will lead to other difficulties. A stone shield upstream of the tie-bolt head would probably be found sufficient protection.

PAPER No. 6.

On Suction Dredgers.

The practice of raising earth by suction dredgers is of comparatively modern date; the Dutch being the first to introduce it on the Amsterdam Canal, where in 1866 they fitted up an old steamer with a suction pipe at the stern of the vessel with a Woodford 3′—6″ centrifugal pump. Improvements have been gradually made and the "Gelderland" suction dredger may now be taken as the most modern type of dredger in Holland.

New South Wales were the next to adopt suction dredgers and their progress has been exceedingly good.

In 1880 Mr. Portus suggested converting two steam hoppers "Neptune" and "Juno" into suction dredgers for clearing navigation channels and reclaiming lands. Pumps were obtained from Messrs. J. K. Smit, Kindirdijk, near Rotterdam. Mr. Darley fixed these pumps, and subsequently developed suction dredging to its present state of perfection in New South Wales.

In 1889 the material raised by all dredgers was three million tons at a cost of $5\frac{3}{4}d.$ per ton.

In 1899 the material raised was eight million tons at a cost of $3\frac{1}{3}d.$ per ton, of this half was raised by suction dredgers at $2\frac{1}{4}d.$ per ton, and the other half by ladder and grab dredgers at $4\frac{5}{12}$ *d.*

The most up-to-date dredger in New South Wales, the "Castor", raised 1,700,000 tons at $\frac{1}{2}d.$ a ton in 1900.

These prices include all expenses excepting interest on capital and depreciation of plant. It may be as well to mention now that 20 cubic feet are taken to be 1 ton; so 1 cubic yard is 1·35 ton.

New South Wales builds most of its own dredgers in the country at Sydney, an example which India may follow with advantage.

America was the next country to take up suction dredging seriously.

As early as 1871 a suction dredger was used in a small way in Florida for deepening bars.

In 1877 the dredger "G. W. R. Bayley" was built with paddle wheels; it can raise about 135 tons an hour.

In 1887 a small dredger was built to raise some 100 tons an hour, to be discharged through 600 feet of pipe; in 90 days it removed 58,000 tons of sand at 9·18 cent. per ton from the Upper Mississippi river. It was regarded as fairly successful.

In 1892 an experimental dredger was constructed with the view of deciding on a design for a dredging boat of "as large a capacity as can be handled in the channel of the Mississippi river at low water with safety and convenience."

It was fitted with forward and trailing suction pipes, and after many experiments the approved dredger * was started at serious work in October 1894; it could discharge 500 to 650 tons through 1,000 feet of pipe in an hour and worked with jets.

* "Alpha."

"Beta" dredger, working with cutters designed by Mr. Lindon Bates, was the next attempt; this was successful on the whole, throwing in its trial, about 6,500 tons an hour through 1,150 feet of pipes. It was soon apparent that cutters were not necessary for sand, so 7 jets were substituted.

In some recent dredgers such as the "Zeta," agitators were tried, but abandoned and jets substituted.

The type now generally adopted in America is a dredger about $160 \times 40 \times 6\frac{1}{2}$ feet, draft $3\frac{1}{2}$ feet, to discharge 1,350 tons through 1,000 feet of pipe per hour, cost $110,000; worked by winches hauling on $\frac{3}{4}''$ wire cables attached to hydraulic hollow piles.

In England practically the only suction dredging that is carried out, is for clearing the Mersey Bar. In 1890 the first suction dredgers were started and Mr. Lyster has gradually improved the designs, finally producing the "Coronation" dredger, which is the finest suction dredger in the world; and is designed to lift 4,000 tons per hour from a depth of 65 feet and with a hopper capacity of 3,500 tons.

The different works for which suction dredgers may be used in India are:—

I.—Removing the bars of harbours. (Rangoon and Madras have already obtained medium sized hopper suction dredgers for their harbours.)

II.—Removing bars of large tidal rivers, such as the Hooghly. (The Calcutta Port have ordered a large suction dredger to be built for them in England.)

III.—Removing bars of inland rivers, and navigation channels such as the Bhagirathi river in Bengal, for which Government is arranging for a suction dredger. Also clearing silt from canals and river channels feeding canals.

IV.—Clearing the mouths of tidal drainages, such as are met with around the Bay of Bengal, including also smaller tidal navigation channels.

As regards No. I, time does not suffice to discuss it. The author collected a few notes on large suction dredgers, which have been printed by the Bengal Secretariat Press, and any one interested can obtain a copy of the memorandum.

Number II concerns the Port Commissioners rather than the Public Works Department.

Numbers III and IV are what concern us more particularly.

The following are a few questions worth considering:—

(a) The best type of dredger to cut a channel 5 miles long with 100 feet base, 5 to 10 feet depth, in a river like the Bhagirathi, which is, say, half a mile broad, with depth of water varying from 3 to 25 feet and velocity from 1 to 5 miles an hour.

(b) The best means of disposing of the spoil.

(c) The best means of handling the dredgers.

(d) The number of suction pipes and nozzles to be used for cutting the channel mentioned above to a fairly level bed.

Suction dredgers may be divided into three main classes:—

I.—Self-propelling hopper dredgers.

II.—Self-propelling dredgers without hoppers.

III.—Non-self-propelling barge dredgers without hoppers.

The objection to class I for river work is, that the size of the vessel increases rapidly with increased hopper capacity; but a dredger with even 300 tons hopper capacity would be useful for small works, for which it might be inconvenient to take floating pipes or hopper barges.

The Americans favour class III, non-self-propelling barge dredgers, but they have a large fleet of tugs available for moving the dredgers, which is not so in India.

Suction dredgers may be fitted with mechanical cutters, agitators or water-jets, the discussion at the conference of the Institution of Civil Engineers in London last year, however, tended to show that ladder bucket dredgers are more suitable than suction dredgers for cutting clay, and that it was not advisable to provide cutters or agitators for suction dredgers, they being costly and never quite satisfactory.

A practical demonstration in support of the above opinion was given by Mr. Lyster, who developed the Mersey Bar dredging. He recently ordered the largest bucket dredger ever made in Great Britain to deal with some clay met with on the Mersey Bar.

Jets may be attached to suction dredgers in certain cases, but for sandy and loamy soil they are hardly necessary.

The different methods practised in the disposal of dredged material are :—

For the self propelling hopper dredgers.

(*a*) To take the material and dump it at sea, or in some place where it will do no harm.

(*b*) To dump the material in a selected place where it can be pumped on shore for land reclamation.

(*c*) To steam to some fixed land pipes, and discharge the spoil on shore for land reclamation.

For all types of dredgers with or without hoppers.

(*d*) To discharge the spoil through floating pipes on to shore or into some place where it will do no harm.

(*e*) To pump the spoil into barges.

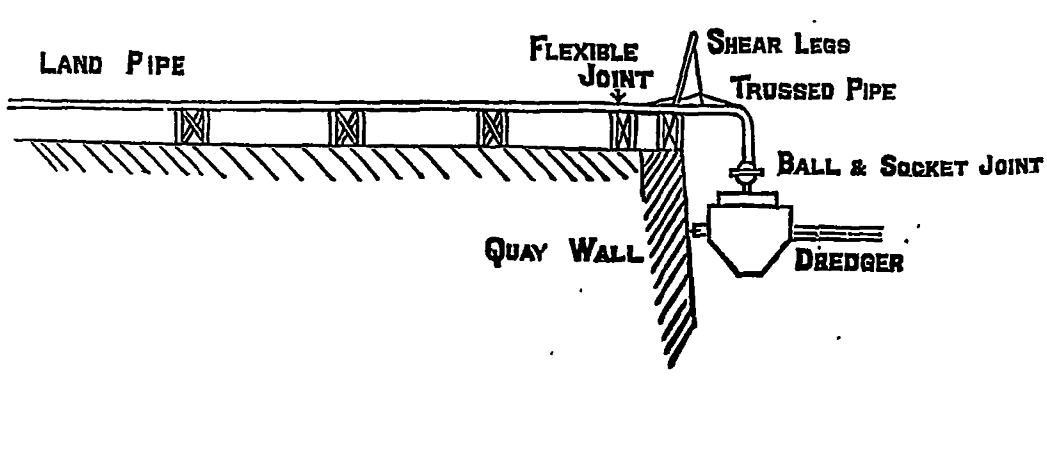
The method (*a*) of dropping the spoil from hoppers is, for large dredgers, the cheapest and the most expeditious provided the lead is not too great, say about 5 miles, but to obtain the best results, the dredger's hopper capacity must be of the maximum size, limited for shallow river work, by the maximum draft permissible.

The method (*b*) of dumping the spoil in a selected site, and pumping it thence through pipes on to land, was originated by Mr. Darley in New South Wales ; it was certainly a novel idea, but has proved to be quite successful and has afforded a solution to the difficulty of reclamation at a reasonable cost, where the areas suitable for reclaiming were far distant from the channel to be deepened.

For pumping the material on shore, the New South Wales Government built the "Castor" dredger, a very powerful one, as it was found that the higher the power of the vessel the more economical are the results. Its engines developed from 800 to 900 H. P. and as much as 1,700,000 tons of sand (340 lakhs of cubic feet) were pumped on to the new lands at Newcastle in one year at the cost of $\frac{1}{2}d.$ a ton (Re. $1\frac{1}{2}$ per 1,000 cubic feet.)

The method (*c*) of the dredger steaming with its load to the end of a fixed pipe and discharging it through that pipe by its own engines, is an excellent one where there is land to reclaim.

The author recently watched this method of unloading at Antwerp. The "Gelderland" mentioned above as one of the most up-to-date Dutch dredgers, has a hopper capacity of 1,500 tons, 30-inch W. I. land pipes were erected running from the quay wall inland for 2,800 feet to the site where the spoil was required for reclamation, while at the quay end there was a length of pipe some 50 feet long, strengthened by a truss, one end of this was joined by a leather and iron flexible joint to the fixed land pipe, the other being turned down and fitted with the upper portion of a ball and socket joint, this end extending beyond the quay wall over the river. The sketch will explain the arrangement.



The river end of this trussed pipe could be raised or lowered by shear legs erected on a fixed platform on the quay wall; the dredger's discharge pipe was vertical with the lower half of a ball and socket joint at its top.

To commence work, the dredger, steamed into position under the overhanging pipe, was moored, the end of the pipe lowered down, and the ball and socket joint bolted up, the whole operation only taking a few minutes. It then proceeded to pump its load down the land pipes, and discharged its load of 1,500 tons through 2,800 feet of pipe in 1⅝ hours, with a shorter lead, the rate of discharge increased, until the time taken was reduced to ¾ hour.

The flexible joint allows the necessary movement to suit varying heights of tide and the varying heights of the dredgers as its load diminishes.

A modification of the two methods described above is also practised in Holland, a moored suction dredger is attached to a land pipe, and hopper barges full of spoil are brought alongside. The moored dredger dips its suction pipe into the barge and proceeds to pump out the contents, a large jet of water meantime playing on the spoil to dilute it sufficiently to enable it to be pumped.

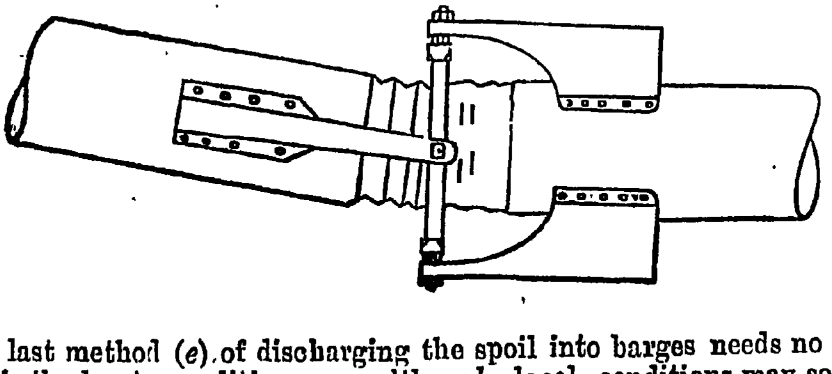
The method (*e*) of discharging the spoil through floating pipes on to the shore or into a part of the river where the spoil will do no harm or may even do good, is the most expeditious one for river work; the pipes may discharge from the side or stern of the dredger and may be single or double, and are generally formed of thin steel or wrought iron coated with asphalte or some similar preservative; their lengths vary from 25 to 100 feet. The pipes are carried on wooden, iron or steel pontoons and are connected by flexible joints.

Two pipes are rather more expensive than one large one, but are more portable; the flexible joints give considerable trouble. In America, rubber joints are adopted 1¾″ thick, with necessary iron framing; Holland and New South Wales adopt leather; wrought iron or steel ball and socket joints are also adopted, but their cost is about three times that of a leather joint which is £5 in Holland.

The "Riga" and "Volga" dredgers built for Russia had floating pipes fitted with "Mailed" flexible joints. The joint is a French invention and made only in France; it is formed of chain mail combined with leather pipes up to 2 feet, cost much the same as O or W steel ball socket joints, but larger pipes are comparatively more expensive; floating pipes are not used in England.

In New South Wales Mr. Portus designed an ingenious gimbal joint which proved very satisfactory, especially in rough water.

The sketch explains its construction.



The last method (*e*) of discharging the spoil into barges needs no description; it is the least expeditious one, although local conditions may sometimes necessitate its use.

As regards the best manner of handling dredgers.

There are three main methods :—

I.—The American system of spuds.

II.—Hauling on mooring piles or anchors.

III.—Running free.

I.—The American spud system has the advantage of cutting a regular channel, the progress, however, is somewhat slow, as the spuds have to be constantly raised and lowered.

II.—Hauling on mooring piles or anchors is expeditious, but it takes a little time to fix the mooring piles or anchors; these when once fixed enable a long reach of channel to be cleared before it is necessary to move them. The dredger can either haul itself along by winches or move with its own engines; side guys may be required for swinging the dredger.

III.—The last method of running free is only applicable for trailing suction pipes; it is specially useful for removing river bars.

As regards the number of suction pipes or nozzles required for cutting a comparatively shallow channel to a fairly even bed, in America as many as six nozzles have been adopted as in the "Beta" dredger, but the consensus of opinion seems to be in favour of one only; Mr. Darley is of opinion that there would be no difficulty in dredging to a level bottom with a single-mouthed suction pipe, and Messrs. Simons & Co., the oldest dredger manufacturers in Great Britain, think that one pipe would suffice, the dredger being swung to cut the necessary width of channel; they consider it difficult to work two pipes at the same time so that each pipe will work to the best advantage, and state that in several dredgers that they have supplied with two pipes, only one was used in practice.

Mr. Portus states that in New South Wales in using grab dredgers converted to suction dredgers, a series of furrows about 50 feet in length, are cut until required width of channel is deepened, or the pipe or dredger is swung.

The author suggests that a suitable dredger for the Bhagirathi or rivers of similar characteristic would be :—

(a) A self-propelling hopper dredger with a hopper capacity of 300 tons capable of discharging 1,000 tons an hour through 1,500 feet of 27″ to 30″ pipe, loaded draft not to exceed 5 or 6 feet, preferably the former. It should be able to pump from its own hoppers, and be capable of steaming 10 miles an hour loaded, and be fitted with winches hauling on fore and aft cables, with side guys for use when required.

It should be provided with a single forward suction pipe, arranged so that it can cut its own way, and alternative side trailing pipe would also be necessary. The cost of such a vessel in England would be about £30,000.

(b) For smaller works some of the dredging plant in India might be converted into suction dredgers on the lines adopted in New South Wales, a full account of which can be found in Mr. A. B. Portus' paper on "Centrifugal Pump Dredging in New South Wales" read before the Engineering section of the Royal Society of New South Wales, 31st October 1896.

It is a somewhat difficult matter to arrive at reliable figures as to the cost of dredging by suction dredgers, but the following is an approximate estimate for India based on figures obtained from England, from New South Wales, and from America :—

	Rs.
Large hopper suction dredgers working in the mouth of rivers, and dumping within a 4-mile lead .	3 per 1,000 c. ft.
Medium sized hopper suction dredgers dumping within a 4-mile lead, or larger barge suction dredgers discharging through 1,000 feet of pipes .	4 ,, ,, ,,
Medium sized suction dredgers discharging through 1,000 feet of pipes	5 ,, ,, ,,
Converted grab dredgers, *i.e.*, small suction dredgers discharging through 1,000 feet of pipes . .	6 ,, ,, ,,

Excluding interest on capital and depreciation of plant.

PAPER No. 7.

The Sone Canals Head Works.

In this paper it is proposed to give a short account of the weir constructed across the river Sone at Dehri to supply water to the Sone Canals irrigating the district of Shahabad in Bengal; describing the method of working the under-sluice shutters of the weir, and the improvements effected recently.

The weir, which is 12,469 feet from abutment to abutment, was started in the year 1870 and finished in 1874.

In the original design, submitted to Government by the Chief Engineer of the East India Irrigation and Canal Company, it was proposed to have "three sets of under-sluices, each 500 feet in length, one set on either flank and one set in the centre of the river."

In each set folding back shutters were to be fixed in bays of 50 feet each.

In the weir as built this proposal was not carried out, as it was finally constructed with 3 sets of under-sluices, each with 22 bays of 20 feet 7 inches; those on the west or Dehri bank measuring, including the piers, 544 feet; the centre 551 feet, and on the east or Barun bank 544 feet.

The length of the weir wall, the crest of which was 8 feet above bed-level, being 5,422 feet on the west and 5,407 feet on the east of the centre sluices; the wall being founded on 6-foot wells sunk 8 feet below bed with another line of 5-foot wells at a distance of 30 feet lower down and sunk to the same depth. The apron was of rubble packing with a slope of 1 in 12 and the front packing a slope of 1 in 3.

The piers of the under-sluices were 4 feet thick, 32 feet long, and 10 feet high above bed-level.

These piers were eventually found not strong enough, as some of them were overthrown in a high flood and the under-sluices were remodelled.

The piers, the masonry of which was strongly bound together with through bolts as rebuilt, were made 6 feet 9 inches thick at Barun and 6 feet 6 inches thick at Dehri; the length in both cases being 32 feet and height 10 feet above bed; the number of vents being reduced in each case to 20, with a width of 20 feet 7 inches.

The centre sluices were reduced to 16 vents of 20 feet 7 inches with 6 feet thick piers and 32 feet long; the remaining 111 feet occupied by the former sluices being converted into a fish ladder with small openings fitted with "*kurries*" or moveable boards fitted in grooves in the piers.

These alterations were carried out between the years 1886 and 1889.

On the crest of the weir, some six or seven years after the completion of the work, were fixed iron shutters 18 feet long and 2 feet high, hinged at the centre of pressure to five tension rods. These shutters fall automatically at a level of 10 feet above bed and are lifted by hand from the back; it taking 20 men to lift them against a 2-foot head.

They are used a great deal in regulating the height of the river, but being easily affected by any slight disturbance in the river owing to storms, etc., are rather troublesome, in that they frequently fall at a lower level than they should.

The gates for the under-sluices were designed by Mr. Fouracres.

They are fully described on page 169 and following pages of Mr. Buckley's book on *Irrigation in India*; but it may be as well to give a short description of them, as it is with reference to the working of them that certain improvements have been made.

In each vent of the under-sluices are fixed an upper and lower shutter. The upper shutter, which is used to close the vent and admit of the lower shutter being lifted and fixed in position, is 21 feet 7 inches long and 9 feet 6 inches high, the piers being recessed to take the extra width of shutter.

It is pivoted at its lower edge, working in two cast-iron gudgeons fixed in the bottom of the piers. At the back are fixed six back-stays consisting of a $2\frac{3}{4}$-inch diameter iron rod working in a $3\frac{3}{4}$-inch diameter wrought-iron pipe; the former being fixed to a cast-iron shoe fastened to the floor of the vent and the latter to a similar shoe on the back of the shutter.

This iron rod working in the pipe is fitted with iron rings and packing so as to act as a piston; while on the under side of the pipe are five $\frac{1}{8}$-inch holes drilled which admit of water entering the pipe when the shutter is down. Three of these holes are placed together at the top of the pipe near the shutter.

When the vents are closed and the lower shutter is in position these upper shutters are laid down horizontally on the floor facing upstream with the telescopic back-stays fully extended; the shutter being held down in position by two vertical rods worked from the piers and fitted with a catch at their lower ends, which engages an angle iron piece fastened to the end of the shutter.

When it is required to raise the shutters and close the vent, after the lower shutters have fallen, the catch is released, the shutter is slightly raised and is brought to a vertical position by the force of the water going through the vent; the water in the pipes which can only escape through the small $\frac{1}{8}$-inch holes as the piston moves up it, acting as a brake to any excessive motion of the shutter.

When the front shutter has been got up, the back shutter is pulled up and the vent closed; the front shutter being laid down on the floor, the space between the two shutters having been first filled with water by opening a valve in the shutter to enable this being done.

The back shutter is slightly shorter than the opening of the vent, 20 feet 7 inches, and is the same height as the piers, 10 feet.

It is hinged to seven $1\frac{3}{4}$-inch tension rods fixed to the gate a little below the centre of pressure with the river at 10 feet on the gauge, the upstream ends being fastened to eye-bolts let into the floor of the vent.

When it is up it is held in position by two chains fastened to bolts in the top beam of the shutter and to hooks fixed on the piers.

In time of flood these chains were connected to the shutter by split links, which gradually opened out as the pressure increased and eventually allowed the shutter to fall.

The falling of the shutter in flood time thus depended on the strength of the iron of the open links, which was very uncertain, owing to the impossibility of making them of uniform strength; and the shutter would fall at any level between 9·50 and 10·00 on the gauge and even at a lower level sometimes.

Improvements and alterations.—The above gives a rough sketch of the weir across the Sone as it used to be. I now propose to describe the recent alterations that have been made.

In the year 1901 half the centre sluices were closed and the other half in March 1902 by a breast wall 7 feet thick and 7 feet above bed, built across the the vents at the lower ends of the piers; an apron of dry rubble stone packing with a slope of 1 in 12 and parallel to the apron of the weir being given at the back of the breast wall.

In the first half of the vents intermediate piers 2 feet thick were fixed, dividing the original vents into 3 bays in which tumbler shutters 3 feet high were fixed; while in the other half these shutters were fixed without any intermediate bays.

A description of these shutters will be given later on.

Hydraulic gear.—The closing of the centre sluices it was calculated would raise, on an ordinary flood, the level of the pool above the weir 2 or 3 inches, which would mean that the under-sluices, fitted as they were with safety or split links, would fall earlier in the flood than formerly.

The irrigation from the canal demands that during the rice season the level in the river should be kept as high as possible, especially at certain times, when any shutter falling would probably produce disastrous results; as some

time would have to elapse before the shutter could be raised, during which time the supply in the canal would be seriously diminished.

It has also been established that, in order to avoid heavy sand entering the canal, it was advisable that the under-sluice shutters furthest from the head sluice should fall first; the first five being kept up as long as possible; the canal being closed as soon as those had to be opened to pass the flood. Although these first five shutters had been fitted with a let-go gear worked from the abutment, it was considered advisable that, seeing the flood level had been raised, perfect control should be had over all the under-sluice shutters as to when they fall.

This control has been obtained by fitting an hydraulic opening gear to each pair of shutters, which is worked from the abutment and enables any back shutter to be released whenever desired.

The arrangement (Plate 16) consists in a 2-inch brass cylinder with a brass plunger fixed on every alternate pier; connected to the cylinder is a ⅜-inch hydraulic pipe, which is let down the lower nose of the pier and along the floor to a pressure cylinder on the abutment, connected to an accumulator weighted to 300lbs. to the square inch.

This pressure cylinder is also in direct communication with a spare pressure pump so that if the accumulator fails, direct pressure may be given from the pump.

On the top of the gates is fixed an inch oscillating rod connected at two points with the longer arm of two ball crank catches, the shorter arm of which is connected to the chains holding up the shutter.

The end of this rod is hooked and engages a small pawl pivoted to a pin fixed in the pier; the whole being held in position by the hooked end of a lever, the other end of which is in contact with the head of the hydraulic plunger.

When it is required to drop the shutter, hydraulic pressure is admitted into the cylinder on the piers, the plunger moves forward and works on the lever, releasing the pawl, which disengages the connection with the end of the oscillating rod and allows the tension on the chains to pull round the bell crank catches and thus release the shutter.

The travel of the plunger is too great to admit of only one shutter being released at a time, and, as it is not of much consequence and saves expense, the shutters are worked in pairs. Three pairs of shutters were fixed with this arrangement in 1901 and the rest in 1902, and have been a great success in every way.

Needle weir.—Control has now been got over the falling of the back shutters; but frequently a flood comes down the river at a period of great demand and all the under-sluices have to be opened and the head sluice closed, in order to prevent a large accumulation of sand and silt in the canal.

In such occasions it is necessary to have the closure as short as possible and to get up the first five shutters so that the canal may be opened; as until this is done the closure must continue.

The hydraulic brakes fitted to the front shutters, which have to be raised first, were supposed to admit of this being done at any level of the river.

Practically this is not the case, and unless it is desired to break all the shutters, the river has to be allowed to go down to 8·50 on the gauge, before it is safe to attempt to raise the shutters.

This means a great deal in a big demand and it is necessary to be able to lift the shutters at any level of the river.

This has been effected by using a kind of needle weir in the vents of the under-sluices shown in Plate 17.

In the side of the piers a 9-inch groove, 5 inches deep, has been cut to a depth of 4 feet 6 inches, from the top of the pier.

In this groove a baulk of timber is dropped and allowed to fall as far as the rush of water through the vent will admit.

In a pocket on the top of the pier and over the groove another baulk of timber is placed.—

Against these baulks of timber needles 6″×3″ and 8 feet long are put, which have the effect of checking the water passing through the bay; the number of needles used varying with the head on which the front shutter is to be raised. The shutter is then raised in the usual way as previously described, and, owing to the cushion formed by the needles, comes up to the vertical position without any shock at all.

Tumbler shutter.—The tumbler shutters fixed on the breast wall closing the centre sluice vents, are an adoption of Mr. Fouracre's shutter described in page 174 of Mr. Buckley's book on *Irrigation in India*.

The shutter (Plate 18) as now made, consists of two parts, the lower part being about one-third the total height of the shutter. Bolted to this is an angle iron piece to which the upper part of the shutter is hinged at about the middle. To the lower part of the shutter at about the centre of pressure for the whole shutter, are fixed three tension rods to which the shutter is hinged, the up-stream ends being fastened to eye-bolts in the breast wall.

At the centre of the shutter, longitudinally, a catch is fixed which engages the lower part of the shutter and keeps the two parts together when the shutter drops, and prevents it from rising in a camel's back as was the case in the original design, the catch being kept closed by the force of the water against the handle.

The shutters are lifted from the front. Iron rods are hooked into catches placed between the vertical battens at the lower end of the upper part of the shutter and the handle of the catch, which is thus released. The upper part is then pulled up, presenting only the edge to the water, which acting on the lower part brings the shutter to a vertical position. The drawing shows the different positions in lifting. The shutters used are 6 feet 10 inches long and 3 feet 6 inches high and could easily be lifted by four men against a 2½-foot head.

There is an objection to them, in that some means is required to admit of men getting on the up stream side unless piers can be built. They are also liable to be damaged, unless protected in some way or sunk a good deal in the weir wall, by trees and brushwood being brought down the river in flood. Some have suffered in this way, but they may, on the whole, be considered as having been a success.

Conclusion.—The improvements described above are due to Mr. R. B. Buckley, late Chief Engineer of Bengal. They have given more control in the working of the weir and enable a fairly constant head being kept in the river to meet the demand for irrigation; the only thing now needed being some means whereby the crest shutters of the weir itself can be controlled, and some method of lifting the lower under-sluice shutters or assisting the manual labour now employed to pull them up. Some system of scouring pipes is also required to clear away the sand and silt that collects in front of the upper shutter when it is raised, and which has to be cleared away by divers before the shutter can be laid down.

The effects of closing the centre sluices can hardly yet be judged. The only thing certain seems to be that it has increased the duration of the floods, but the inconvenience of this is counteracted by the improvements made in working the under-sluices.

PAPER No. 8.

The construction of weirs in Rohilkhand.

The Kicha River in Rohilkhand has always given great trouble to engineers. Its volume rises from 20 cusecs in the hot weather, to 80,000 in the monsoon.

Like all Tarái streams, it also has a habit of partially leaving its valley and spilling into adjacent drainages. Similarly the neighbouring streams frequenty overflow their basins, and thus swell the Kicha volume to unmanageable dimensions.

Since 1840, three attempts have been made to provide the irrigation channels of this river with a permanent weir, but when I first visited the site in 1899, there was nothing left but the head of the canal, right and left revetments, and fragments of the last weir which had been shuttered in 1889, and further smashed in 1897.

It was certainly a very depressing spectacle, and if I could, I would have left it altogether, and sought a better site elsewhere. This, however, was not practicable, as about a lakh of rupees had been recently spent in training the river into its present course. Several new groynes had been constructed, and when they had proved their efficiency, it was proposed to restore the weir in some form or other.

The old works consisted of a superstructure of masonry on a single row of wells, with aprons of pitching, cribs, and piles, etc., Plate No. 20. I take it that they failed on account of the treacherous quicksand nature of the Kicha bed, which has always been dreaded by the *mahouts* of Rohilkhand.

As the revetments and head of canal were in good order, and had plentiful protection all around them, it was plainly desirable to utilize the old site if practicable Wells were, I think, impossible on account of the presence of *débris* of all sorts, not to speak of buried trees, etc., brought down in the floods.

The crest length of the old weir was well known to be insufficient, as a maximum flood was calculated to be 80,000 cusecs, and had to pass through a waterway of 300×17 feet, which would generate a velocity of $\frac{80,000}{5,100} = 15 \cdot 6$ feet per second. To meet these conditions I decided to adopt the following unusual course :—

- (*a*) Provide an escape outlet for high floods through the marginal bund, about 1 mile above the weir. Waterway to be $300 \times 8 \cdot 5 = 2,550$ square feet.
- (*b*) Replace the old weir by a new one, which was to be laid on 10 rows of crates ($10 \times 10 \times 5$ feet). *Vide* Plate No. 21.

The reasons for these conclusions are :—An examination of the plan of the weir on Plate No. 19 shows that heavy flood passing round the sharp bend above the weir will

- (*a*) always be liable to punish the groyne heads ;
- (*b*) arrive at the weir in such an angry swirling condition that deep holes are sure to be scoured out in the vicinity.

Hence we ought not to force a high flood round the curve but should escape a large proportion of it over the bye-wash at A. This suggestion is advantageous :—

- (*a*) because the action of the escape will take much of the fury out of the flood that passes through the training works ;
- (*b*) because the escaping volume will throw a cushion below the weir and prevent deep scouring action ;
- (*c*) because the provision of the bye-wash is closely adapted to the usual course a heavy flood takes in a Rohilkhand river. When a valley is surcharged, the swollen waters invariably find outlets

by spilling over bends or even over a watershed into neighbouring basins. I may add, however, that the presence of a railway to bring in boulders, and *sal* poles, made the scheme a convenient one.

The byewash escape was completed before the floods of 1903, but an epidemic of cholera scattered the labourers, when only 6 rows of the weir crates were in position. The maximum flood rose to 10·00 and did not move a crate, which was satisfactory. Only a film of one foot passed over the byewash and did no harm. The design of this work is extremely light and cheap. It was purposely adopted, as the work will never be exposed to action for more than a few hours at a time. It would have been easy to have built a much safer structure, but I was anxious to keep within limits. If, moreover, it succeeds, we shall probably utilize this device of dissipating the force of floods, on other rivers under similar circumstances.

The weir works were completed in time for the floods of 1904, and are of great use in securing the rice crop. I visited the site when the six rows of crates were laid bare, and the foundation pit for the remaining four rows was dug out.

I was glad to find the barrage made in the previous year was absolutely watertight, on account of the Kicha mud having penetrated all the crevices between the boulders.

I pleaded against the use of wells in this case, although they were recommended by several engineers, because their construction is a most tedious and unsatisfactory form of making the foundation of a dam. In this particular case the difficulties in sinking a well would have been practically insuperable on account of the *débris* that was certain to be met with. The slowness of well sinking is a great objection in Rohilkhand, where the working season is very short, and it is desirable that any work undertaken should be finished up to a safe stage in the same season. In truth, it may be said that all the weirs in this province, which depended on brick cylinders, have been singularly unfortunate. At this meeting the case of the trouble with the Narora and Hindan weirs will be discussed. The Kicha weir that we have now replaced, also depended on wells. The Kylas weir had no less than four rows of wells, and has failed several times. It is not contended that a sound structure cannot be built on wells, and I myself have constructed some with infinite trouble; but I contend that the money is better spent on broad and shallow foundations, as was done on the Okhla weir. In a recent* number of "Public Works" I saw that Sir W. Wilcocks described the successful construction of a barrage on the Nile, the sub-structure of which consisted of ordinary rubble tipped into the bed of the river. He added that the Nile mud had penetrated all the crevices, and rendered the work quite watertight.

* May 1904, page 195.

PAPER No. 9.

Financial results of the Deoband Branch, Upper Ganges canal.

The construction of the Deoband branch was first proposed in 1861 by Mr. Login, to benefit a tract of country which had suffered severely in the drought of 1860-61; it was not, however, until 1878, that this work was begun under an estimate prepared by Mr. A. Grant, Executive Engineer, and sanctioned for Rs. 4,44,219 for works.

The branch takes out of the right bank of the Ganges canal at mile 22, and runs generally in a south westerly direction across the Sila and Kali Naddis, and then from mile 18 it turns southward along the watershed lying between the Kali and Hindan Naddis.

The main line is 51 miles 4 furlongs in length, and there are at the present time 77 miles 4 furlongs of distributary channels, and 71 miles 7 furlongs of minor channels; including the improvement of the Sila Naddi, there are 104 miles of drainage cuts.

The canal was originally designed to carry 400 cusecs as a maximum, owing, however, to recent berm cutting and widening, it can carry 500 cusecs as a maximum. It was expected to irrigate 29,000 acres in the rabi, and 27,000 acres in an ordinary kharif, and 30,000 acres in a year of drought. The average of the last 5 years shows that it has irrigated 36,164 acres in rabi, and 26,795 acres in kharif, or a total of 62,959 acres as against an estimated total of 56,000 acres.

The total capital outlay up to the end of 1902-1903, was Rs. 4,89,962 and the total revenue assessed to the same period was Rs. 38,14,855. Deducting expenditure under revenue, cost of revenue establishment, tools and plant, and all capital charges, together with simple interest at the rate of 4 per cent., the net revenue earned to the end of the same period was Rs. 23,80,112, or nearly 4·8 times the capital expenditure.

The average percentage of return during the 5 years ending 1902-1903, has been 36·1 per cent, as compared with 21·8 per cent. of the previous 5 years.

The estimated profit was Rs. 1,00,000 or 23 per cent. of the capital outlay.

The actual revenue assessed has risen from just over 1 lakh in 1881-82, to 2·8 lakhs in the maximum year of 1899-1900, or to an average of 2·4 lakhs during the last quinquennial period. The good results obtained during the last 5 years may be attributed to a series of dry years when water was in much demand for irrigation, and to an improvement of the supply given at the head.

The total length of canal and irrigation channels was in

	Mile.	Furlong.
1893-94	202	3
1898-99	207	1
1902-03	214	0

So that there has been no considerable extension of irrigation during the last decade to tracts previously without irrigation.

The average kharif irrigation of the last 5 years was 26,800 acres principally sugar and rice, the average areas under these crops being

	Average.	Maximum.
Rice	9,500	10,800
Sugar-cane	13,500	15,000
TOTAL .	23,000	25,800

while the kharif area has increased from 22,000 to 26,800 acres in the two last quinquennial period or by 4,800 acres only, on the other hand the rabi area has increased from 21,500 to 36,000 or by no less than 14,500 acres. The rabi area of course is more or less dependent on the rainfall.

From the appended statement it will be seen that the duty, varying with the season of course, is about 100 acres per cusec in kharif, and 200 acres per cusec in rabi.

The annual rainfall of this tract of country varies from 19 to 45 inches, and has a mean of 31 inches.

For the United Provinces it may be considered a wet tract of country as compared with tracts further south, and more distant from the hills.

Owing to a better and more assured rainfall, and also to a greater suitability of the soil, it is always in such tracts in these Provinces, that there is a much more extensive cultivation of both sugar and rice than elsewhere.

These two crops which pay first class rates take water at some period, whatever the rainfall, and it is in a large measure due to the annual steady revenue derived from their irrigation, that the Deoband branch has proved such a success.

The revenue derived from the irrigation of rabi crops is on the other hand more fluctuating, as it depends considerably on the season, but not to the extent one might suppose from a consideration of the total rabi rainfall, since this rain often falls at uncertain periods, and not always when most required.

It is of course in years like 1899-1900, when the monsoon closes early and the rabi crops cannot be sown without water, that both canal revenues and the cultivators benefit most.

DEOBAND BRANCH.

Statement showing average area irrigated and revenue assessed for quinquennial periods.

Period.	Number of years.	Average area irrigated in acres.			Average revenue assessed.
		Kharif.	Rabi.	Total	
1879-80—1882-83 . .	4	7,180	10,346	17,526	60,574
1883-84—1887-88 . .	5	15,063	18,736	33,799	1,33,487
1888-89—1892-93 . .	5	18,416	21,062	39,478	1,60,321
1893-94—1897-98 . .	5	21,991	21,426	43,417	1,72,667
1898-99—1902-03 . .	5	26,795	36,164	62,959	2,48,036

DEOBAND BRANCH.

Statement of kharif and rabi duties.

Years.	Number of running days of Ganges Canal at head.	Total daily consumption including Right Main Distributary.	Kharif duty.	No. of running days of Ganges Canal at head.	Total daily consumption including Right Main Distributary.	Rabi duty.	REMARKS.
1899—1900 .	149	39,564	111	182	40,794	204	
1900—01 . .	107	34,963	78	154	12,248	339	
1901—02 . .	127	35,956	99	182	37,218	212	
1902—03 . .	118	28,536	109	171	28,318	226	

NORTHERN DIVISION, GANGES CANAL.

Statement showing area irrigated, Revenue assessed, and percentage of return on Capital Outlay for the Deoband Branch excluding the Right Main System.

1	2			3	4			5	6	7	8	9	10	11
Year.	Area Irrigated in acres.			Revenue Assessed.	Expenditure on Revenue.			Net Revenue Columns 3 and 4.	Capital Outlay during the year.	Capital Outlay to end of year.	Simple interest at 4 per cent., on capital outlay to end of previous year plus ½ outlay during the year.	Net Revenue less simple interest.	Percentage.	Average percentage of quinquennial periods.
	Kharif.	Rabi.	Total.		Extensions and Improvements.	Repairs.	Establishment and Tools and Plant.							
Probable expenditure on constructing the Sedhaoli Distributary 21·7 miles at 2,000									43,750					
Previous to 79—80									3,44,921	3,88,671				
1879-80	1,978	2,814	4,792	19,526	...	896	19,186	18,364	63,185	4,51,856	18,074	290	·06	
1880-81	2,175	4,699	6,874	21,920	...	3,000								
1881-82	9,486	18,357	27,843	1,00,283	677	6,840	22,436	70,330	12,461	4,64,317	18,321	52,007	11·2	
1882-83	15,081	15,514	30,595	1,00,567	575	6,758	18,383	74,851	1,042	4,65,359	18,594	56,257	12·1	5·84
1883-84	15,213	23,865	39,078	1,40,609	...	5,721	26,395	1,08,493	3,618	4,68,977	18,687	89,806	19·1	
1884-85	15,699	10,342	26,041	1,16,278	1,240	3,778	30,679	80,581	2,627	4,71,604	18,812	61,769	13·1	
1885-86	14,718	25,904	40,622	1,49,521	86	3,324	32,894	1,13,217	3,376	4,74,980	18,932	94,285	19·8	
1886-87	14,891	15,785	30,676	1,26,739	...	4,958	34,219	87,562	1,317	4,76,297	19,026	68,536	14·3	
1887-88	14,794	17,785	32,579	1,34,287	...	6,014	33,873	94,400	320	4,76,617	19,058	75,342	15·8	16·42
1888-89	17,065	11,662	28,727	1,31,118	1,273	7,439	33,956	88,450	6,518	4,83,165	19,196	69,254	14·3	
1889-90	16,597	29,898	46,495	1,67,072	766	7,764	37,878	1,20,664	1,753	4,84,918	19,362	1,01,302	20·8	
1890-91	19,273	15,548	34,821	1,50,792	1,841	6,103	36,190	1,06,658	...	4,84,918	19,397	87,261	18·0	
1891-92	19,595	25,192	44,787	1,80,390	826	6,898	41,490	1,31,176	...	4,84,918	19,397	1,11,779	23·0	
1892-93	19,549	23,009	42,558	1,72,230	2,947	9,753	43,059	1,16,477	1,312	4,86,230	19,423	97,054	19·9	19·20
1893-94	19,511	18,961	38,472	1,69,389	4,009	9,203	40,792	1,15,385	1,160	4,87,390	19,472	95,913	19·7	
1894-95	21,033	661	21,694	1,26,281	1,178	13,481	42,935	68,687	...	4,87,390	19,496	49,191	10·9	
1895-96	18,523	23,347	41,870	1,73,974	...	7,567	33,107	1,33,300	72	4,87,462	19,497	1,13,803	23·3	
1896-97	24,763	36,914	61,677	1,69,363	2,155	6,060	26,054	1,35,094	2,500	4,89,962	19,548	1,15,516	23·5	
1897-98	26,123	27,245	53,368	2,34,329	...	9,602	40,379	1,74,348	...	4,89,962	19,598	1,54,750	31·5	21·78
1898-99	24,106	27,341	51,447	2,16,881	2,396	9,231	41,207	1,64,047	...	4,89,962	19,598	1,44,449	29·4	
1899-1900	29,638	45,656	75,294	2,82,328	4,471	10,914	39,526	2,27,417	...	4,89,962	19,598	2,07,819	42·4	
1900-1901	25,606	26,819	52,425	2,21,840	562	8,783	31,058	1,81,437	...	4,89,962	19,598	1,61,839	33·0	
1901-1902	28,028	43,392	71,420	2,61,346	484	8,534	41,429	2,07,899	...	4,89,962	19,598	1,88,301	38·4	
1902-1903	26,596	37,612	64,208	2,57,786	4,323	10,306	40,000	2,03,157	...	4,89,962	19,598	1,83,559	37·5	36·11
Total	440,041	528,322	968,363	38,14,855	29,809	1,72,927	7,90,125	28,21,994	4,89,962	1,14,54,803	4,41,882	23,80,112	491·06	99·38

PAPER No. 10.

The financial results from Certain Drainage Cuts.

The tract of country in the Bulandshahr and Aligarh Districts, which lies between the Mat Right Branch and the Barauda Distributary from mile 18 of the former to their Junction, and extends to the east of the Barauda Distributary between miles 23 and 31, and again beyond the tail of the Barauda Distributary for some miles still on the east of the Mat Right Branch, consists of depressed shallow valleys with apparently no proper outfall for the drainage.

This drainage had, however, a tendency to run southward and find its way as best it could eventually into the Karon and Patwaya Naddis, affluents of the Jumna.

The generally accepted opinion was that the Mat Right Branch obstructed the flow of drainage but this was not so, for as a matter of fact, this Branch with but few slight exceptions, had been taken along the best watershed, over which water could pass only when the flooding of this valley was extremely heavy.

It is extremely doubtful whether such flooding ever took place before the advent of the canals; it is far more probable that in those days when irrigation was effected from wells the rainfall was all absorbed in these depressions, and there was practically no flooding or moving on of the drainage. Constant irrigation from the Canal had, however, raised the spring level and rendered the soil less capable of absorbing the rainfall, whilst every cycle of years of heavy rainfall had brought the necessity of draining this tract prominently to notice, as far back as the year 1876.

The problem of draining this tract was complicated by the non-existence of any really proper outfall, and the fear that drainage by flooding might result elsewhere.

After a very careful enquiry, it was decided that the proper way to deal with the drainage was to take it partially into the Karon Naddi, but the greater portion into the Patwaya by crossing the watershed on which the Mat Right Branch is situated, and fitting the Patwaya to receive the extra volume.

The Parauri, Gomat and Pisawah Drains, which form the subject of this paper, are part only of the general scheme for draining this tract of country.

The Parauri Drain was intended to drain the land lying along the east bank of the Mat Right Branch above mile 29 into the Patwaya Naddi.

The Main Drain was completed in 1894, but was extended upwards in 1897, when also the Mahaballipur and Manchar Branches were constructed.

The Main Drain has a length of 16m. 6f. and it has 3 branches of a total length of 8m. 7f., making a grand total of 25m. 5f.

The total catchment area is 33 square miles.

The total cost of this drainage system was Rs. 34,911.

The Pisawah Drain was constructed to drain a Jhil lying to the North of Pisawah, and east of mile 39 of the Barauda Distributary, by passing the floods under the Barauda Distributary at mile 30, and then across the tract to mile 33 of the Mat Right Branch, picking up all the drainage lying to the South of a line drawn from mile 28 of the Barauda Distributary to mile 30 of the Mat Right Branch, and carrying it down to the Patwaya Nala.

This drainage system was completed in 1895, at a total cost of Rs. 24,200.

The main drainage is 8m. 3f. in length with branches 7m. 4f. in length making a total of 15m. 7f.

The total catchment area is 24 square miles.

The Gomat Drain was intended to drain off the water in the large Sujanpur-Gomat depression, which was flooded by water coming down between the Mat Sub-Branch and the Barauda Distributary, and crossed the Sofa Escape

(which although it acted as a drain was of small capacity and moreover inefficient when the Karon Naddi was in flood) and the Tappal road.

This drain was completed in 1901.

It has a length of 10m. 2f. and a catchment area of 21 square miles.

It was constructed at a cost of Rs. 7,231.

In connection with the Parauri and Pisawah Drains, the Patwaya Nala was improved below the outfall of the former at a cost of Rs. 23,788.

The total cost of these 3 drains together with the improvement of the outfall was Rs. 90,131; this is equivalent to Rs. 1,155 per square mile of catchment area.

As far as can be ascertained from records, the first mention of severe floods in this tract is in 1873, when considerable damage was done and again in 1874, but to a less extent, and Major Marindin states in his report of 1875, that remissions had several times been granted.

This tract again suffered severely in the wet years of 1884, 1885 and 1887; 1887 was a year of very heavy rainfall following on three years of high rainfall, and the damage done by floods was very great.

The Collector of Aligarh reported that in September 1887, the country was one sheet of water in every direction, and many villages were completely cut off from all access for nearly a fortnight, and that even in February of the following year, large lakes from ⅓ to ½ mile in extent still covered what was formerly cultivated land: the ground too had become saturated in many of the villages and the spring level had risen within 3 feet of the ground.

A large area of Kharif crops was totally destroyed, and much of the rabi remained unsown owing to the saturation of the soil.

Considerable remission of land revenue and water rates were made, and the loss to Government both direct and indirect was very great.

Owing to repeated damage by floods, the cultivated kharif and irrigated rabi areas fell off considerably and fever decimated the villagers and several thousand head of cattle died.

Severe damage to this tract occurred again in 1890, owing to floods, which caused destruction to the kharif crops, loss of cattle from eating the rank grass which sprang up: much of the rabi could not be sown, or if it was sown, it was too late to produce a good crop.

This brief account of damage from floods in this tract in 1887, and again in 1890, will show how serious the matter had become.

In these two years alone, Rs. 12,741 of the land revenue was remitted, together with Rs. 7,347 occupier's and owner's rate, making a total of Rs. 20,088.

I have not been able to trace the amount paid in remissions on account of damage by floods in years previous to 1887, but I estimate that the total direct loss to Government on account of land revenue, owner's and occupier's rates, which had to be remitted, could not have been less than Rs. 30,000, an amount which represents 33 per cent. of the cost of the drainage system.

In addition to these direct losses, there was an indirect loss due to the falling off of the canal irrigated area, especially of the rabi area.

The amount of this loss is difficult to estimate, but from a consideration of the rainfall figures, and the irrigation figures of the past 32 years, I find that in round figures, the increase of irrigation since the construction of these drains, which may fairly be attributed to their construction, to be as folows:—

	Acres.
Pisawah Drain	1,000
Gomat Drain	2,000
Parauri Drain	1,000
TOTAL	4,000

The average occupier's rate per annum in the Bulandshahr Division is Rs. 3 per acre, so that this represents an increase in Canal revenue alone of

Rs. 12,000 per annum, equivalent to a return of 13·3 per cent. on the Capital cost of the Drainage scheme.

It lastly remains to be seen as to how far the land revenue has benefited by the construction of these drains.

The settlement of the Aligarh District, in which most of these villages lie, has just been concluded, and there can be no doubt that, but for the construction of these drains, the land revenue must have been reduced owing to the condition of these villages, which was steadily deteriorating from the causes mentioned.

Instead of a reduction I find there has been an enhancement, varying from Rs. ·25 to ·95, and averaging Rs. ·5 on the cultivated area.

The whole of this enhancement cannot of course be attributed to the construction of these drains, but it is no doubt largely due to this, for the benefits the land has derived are as follows :—

1—Protection from flooding and consequent loss of kharif crops.

2—Increase of the cultivated rabi area and better rabi crops.

3—As a consequence of the former extended irrigation.

4—Increased healthiness of both men and beasts.

After a careful consideration of the facts of the case, and the examination of the last settlement records, and after consultation with the District Officers, I consider the land revenue to have benefited to the extent of Re. 1 per acre of the cultivated area in those villages which suffered most severely from floods, and Rs. 0 25 per acre generally on the cultivated area in other villages within the catchment area.

In this way I estimate the benefits to land revenue to be as follows in the case of :—

	Rs.
Piawah Drain	6,500
Gomat Drain	7,500
Parauri Drain	4,000
Total Rs.	18,000

which sum is equivalent to a return of 20 per cent. on the Capital cost of the Drainage scheme.

Including canal revenue it may, I think, be safely stated that the net return to Government from the construction of these drains has not been less than 35 per cent. of the total expenditure incurred, if the benefits due to improvement of the Patwaya Nalah are, as they should be, included.

There are no figures available to estimate this with any accuracy, but undoubtedly there is less liability of damage to the kharif crops sown on the marginal lands, and in years of high rainfall, these lands have been rendered more fit for rabi sowings.

Statement of rainfall.

Years.	Parauri Drain.	Pisawah Drain.	Gomat Drain.
	Parauri.	Balanpur.	Sujanpur.
1878-1879	2·90	4·10	
1879-1880	43·30	32·60	
1880-1881	24·00	19·50	
1881-1882	22·40	24·60	
1882-1883	21·00	15·60	
1883-1884	8·90	16·30	
1884-1885	42·60	35·50	
1885-1886	39·65	34·35	
1886-1887	29·85	22·50	
1887-1888	45·45	42·95	
1888-1889	25·50	24·70	24·30
1889-1890	19·30	21·50	20·30
1890-1891	32·60	37·30	33·90
1891-1892	24·65	28·95	30·40
1892-1893	30·10	29·45	37·60
1893-1894	20·95	25·10	19·90
1894-1895	33·20	38·00	26·60
1895-1896	10·00	17·90	19·00
1896-1897	28·10	34·60	28·20
1897-1898	32·40	26 80	33·20
1898-1899	19·00	14·50	17·70
1899-1900	20·30	20·20	16·50
1900-1901	36·10	35·00	35·90
1901-1902	18·10	15·50	12·90
1902-1903	19·50	20·80	25·00
1903-1904	15·40	21·50	21·50
Total .	665·25	649·30	403·40
Average . .	25·59	24·99	25·21

PARAURI DRAIN.

Statement showing depths of Wells to Spring Level.

Years.	Annual Rainfall.	No. 1.		No. 2.	
		May.	September.	May.	September.
1890		632·30	633·30	632·70	633·70
1891		631·90	636·20	631·70	638·50
1892		631·30	637·90	634·30	633·60
1893		632·16	634·31	632·89	636·69
1894		631·46	637·41	632·34	638·79
1895		631·01	630·46	634·49	633·79
1896		631·01	630·46	632·72	630·69
1897		631·46	632·46	630·29	631·69
1898		632·46	632·21	631·89	630·04
1899		631·01	630·31	628·39	628·19
1900		632·26	...	635·89	...
1901		632·96	634·40	634·07	636·47
1902		630·57	631·79	634·47	637·57
1903		631·45	632·85	632·27	633·37

Note.—No. 1 well is at Parauri.
No. 2 well is at Jehangirpur.
These wells do not appear to have been affected by the drainage cut.

The Pur Drainage Scheme, Muzaffarnagar District.

The Rehi and Barla-Chappar drains were constructed in 1875-78, to drain the villages of Medpur, Rehi, Chappar and Barla with the object of remedying damage done by percolation, and of improving a tract of country with a naturally defective drainage.

Subsequently in 1896-98, this drainage system was extended to Pur in order to improve the country on the left bank of the Right Mohammadpur Distributary.

The scheme included the widening of the old cuts, and is now known as the Pur Drainage Scheme.

The Pur Main Drain Cut, starting from tanks North of Pur, runs southward, and crossing the road from Deoband to Bijnor and Right Mohammadpur Distributary, joins the old Barla-Chappar Cut near Parai, and has its outfall in the Kali Naddi.

The main drain of this system is 14 miles 3 furlongs in length, and has branches aggregating a length of 25 miles 1 furlong, making a total of 39 miles 4 furlongs.

The total cost of the scheme has been Rs. 72,879, and the drainage catchment area is 38·5 square miles. This is equivalent to Rs. 1,893 per square mile of catchment area—a high rate.

This tract of country is generally of a sandy nature with spring level originally at a great depth below the ground; it required originally no outlet for its drainage, the rainfall of ordinary season being ordinarily absorbed.

The water level was originally according to Mr. Cadell, who revised the settlement in 1870-74, at a depth of 60 feet in the lower part of this tract and 80—100 feet in the higher part, and in his time it had risen to 20 feet and 50 feet respectively, and has since risen higher.

This rise in spring level was due partly to the high level at which the Right Main Distributary and its branches had been constructed, and partly to the obstruction to drainage, caused by numerous embanked guls and distributaries, and the fact that the natural absorbtive capacity of the soil had been reduced by irrigation.

It was in order to improve the condition of this tract of country, which was steadily deteriorating, that the Pur Drainage Scheme was undertaken.

The total culturable area at the time of settlement in 1892, and before the construction of the drain, was 22,492 acres; it is now 23,269 acres, an increase of 777 acres: the total cultivated area has risen in the same period from 20,890 acres to 21,820 acres, an increase of 930 acres.

There has, therefore, been no marked increase either in the cultivated or culturable area since the drains were constructed.

A comparison of the average irrigated areas of 5 years from 1885-89, preceding the construction with those of 1899-03, subsequent to the construction of this drainage scheme, shows that the rabi irrigated area has risen from 4,867 to 6,103 acres, an increase of 1,236 acres, and the kharif area from 5,244 to 6,330 acres, an increase of 1,186 acres, in other words, there has been an increase of 2,422 acres in the annual irrigated area since the construction of the drain.

As, however, it is the rabi area which is most likely to be affected by the saturation of the soil from flooding, I have compared the average area under rabi for the years 1893-95 and 1902-04, the figures are 13,148 and 13,454 acres respectively, showing a merely nominal increase of 306 acres.

A comparison of the rent rolls of the villages will, however, I think, show most clearly the improvement that has taken place in this tract since the settlement of 1892.

The rent rolls of these villages at the time of settlement were Rs. 55,309, and they are now Rs. 93,309, an increase of Rs. 38,000 or 69·1 per cent.

With the exception of Barla, the rent rolls of all the villages show an increase.

These latter figures indicate, I think, clearly that the drainage of this tract has resulted, not so much in an extension of cultivation, as in a great improvement in the productiveness of the soil; land which used to be very precarious is now secure of a decent crop. One result of this will no doubt be a considerable rise in revenue at the next settlement, and the Collector of this district has expressed his opinion that the land, roughly speaking, could bear an assessment 50 per cent. higher than the current one.

The land revenue is Rs. 48,192, and the cultivated area at the time of settlement was 20,890 acres giving an incidence of Rs. 2·3 per acre.

An increase of 50 per cent. in the assessment would be equivalent to an increase of Rs. 1·1 per acre on the cultivated area.

I think it may fairly be said that Re. 0·6 per acre may be attributed to the drainage of this tract, which on the present cultivated area is equivalent to Rs. 23,820 × 0·6 = Rs. 13,092.

In addition to this, I think, that 50 per cent. of the increase in the irrigated area or 1,200 acres may be taken as due to the effects of the drainage.

Since the occupier's rate in the Northern Division is about Rs. 4 per acre, an additional revenue of Rs. 4,800 from water rates may be attributed to the drainage cuts also.

The total gain to Government may, therefore, be estimated at Rs. 17,892 per annum, or Rs. 18,000 in round figures, equivalent to a return of 24·7 per cent., or in round figures 25 per cent. on the capital expenditure incurred.

This, from a careful examination of facts and figures, I consider a fair and reasonable estimate.

No remissions of land revenue in this case are recorded to have been granted on account of damage by floods.

Appended is a statement of the rainfall at Deoband, a station not within the catchment area, but about 10 miles from its centre, also a statement showing the depths of water below ground level in certain wells, *vide* plan within the catchment area.

It is much to be regretted that observations of these wells have been discontinued since 1898, otherwise something might have been learnt of the effect of drainage on the spring level in this tract. The figures merely show, as compared with the facts stated in paragraph 4, to what an extent the spring level had risen.

DEOBAND BRANCH.

Annual Statement of Rainfall at Deoband.

Station.	Year.	Kharif.	Rabi.	Total year.
Deoband	1883	23·20	6·10	29·30
	1884	45·50	2·10	47·60
	1885	32·10	6·20	38·30
	1886	41·10	1·80	42·90
	1887	36·10	1·80	37·90
	1888	32·90	2·10	35·00
	1889	34·40	4·70	39·10
	1890	35·30	2·30	37·60
	1891	42·40	5·10	47·50
	1892	27·23	1·30	28·53
	1893	27·21	4·60	31·81
	1894	37·15	11·87	49·02
	1895	42·40	6·13	48·53
	1896	19·06	1·64	20·70
	1897	24·82	0·87	25·69
	1898	29·43	6·19	35·62
	1899	19·19	0·91	20·10
	1900	32·44	4·00	36·44
	1901	26·40	5·46	31·86
	1902	34·66	0·28	34·94
	1903	25·67	1·62	27·29
Average of 21 years		31·84	3·67	35·51

POR DRAINAGE SCHEME.

Statement showing depths of Wells to Spring Level.

Years	Annual rainfall.	No. 1		No. 2		No. 3		No 4		No. 5		No. 6	
		May.	September.	May.	September.	May.	September.	May.	September.	May.	September.	May	September.
1890	37·60	6·7	3·2	6·7	6·0	12 0	7·4	9·5	6·1	12 6	7·0		
1891	47·50	7·0	4 5	7·6	6 1	15·0	4·5	11·8	3·5	14·1	5·2		
1892	28·53	7·2	6 0	11·6	6·2	12 8	6·0	10·5	4·0	14·7	7·4		
1893	31·83	7·0	5·7	12·6	8 0	14·8	8 2	12·3	6·5	16·4	13·0		
1894	49·02	8·1	5·8	15·2	8·0	14·0	4·8	12 8	3·2	16·2	12·0		
1895	48·53	6·7	3 9	10·2	5·2	13·2	6·0	12·0	5·9	15 5	8·2		
1896	20·70	6·5		13·5		12·6		12·9		14·1			
1897	25·69	7·7	5·3	10·5	10·5	11·6	6·9	9·6	5·0	14·4	14·2		
1898	35·62	7·8	9·1	21·0	14·0	14·2	6·7	12·2	5·3	18·5	11·5		
1899	20·10												
1900	36 44												
1901	31·86												
1902	34·94											824 4	824·7
1903	27·29											820·4	823·2
1904												820 6	

PAPER No. 11.

On Silting Operations to strengthen high Canal Banks.

Reason for silting operations.—When large canals cross low ground, with the bed very little in soil, or it may be considerably above ground level, large quantities of soil are required for the construction of the banks; the banks require to be made specially strong where abnormally high and this involves digging formidable borrow-pits either inside or outside the canal, and these are objectionable in the latter case in low ground.

Canal water, silt laden.—In all canals in the Punjab, to which alone this note refers, the water at all seasons of the year, but particularly in the hot weather, carries a certain amount of silt which usually varies from $\frac{1}{1000}$th to $\frac{1}{5000}$th part of its volume. To separate the greater part of this silt from the water, it is only necessary to lessen the velocity throughout a distance varying from 1,000 feet to 5,000 feet, when the coarser part will settle down and remain at the bottom of the channel. The means adopted for lessening the velocity is to increase the sectional area by some of the expedients described below.

System of silting operations in use in the Punjab.—The systems which have been followed at different times in the last thirty years, have been as follows, in the order in which they have been adopted, which is also that of their relative merits as regards efficiency and economy:—

(*a*) Procuring as much of the spoil as possible from deep bed borrow-pits dug between the canal banks, the balance being got from outside borrow-pits; *vide* Fig. 1 on Plate 22. It was the practice in past years to keep the canal banks at the normal distance apart. This was known as the system of bed borrow-pits.

(*b*) Constructing additional banks alongside the boundary roads with cross banks at intervals of 500 feet to 1,000 feet apart, the series of compartments thus formed being silted up by giving inlets and outlets at the upstream and downstream ends, respectively, and allowing a certain part of the canal supply to pass through the silting reaches formed as just described. This is usually known as the "in and out" system of silting reaches, and is shown in Figure 2.

(*c*) Constructing external parallel banks as above described, with cross banks at 4,000 feet to 5,000 feet intervals, and with head inlets and tail outlets, the whole canal supply being passed through one silting reach at a time; all that should be kept open under this system. The canal channel opposite the silting reach in operation is closed off at head and tail to obviate it being silted up. (Figure 3). This is known as the long reach system.

(*d*) Constructing the canal banks some distance back from the normal section of the canal channel, inducements being given for silt being laid down internally on the berms. (Figure 4). This is known as the system of internal silting.

These systems, especially the last one, will now be dealt with in detail.

Bed borrow-pits system, Figure 1.—The pits never silted up fully to bed level; abnormal side silting on the berms except at the cross bars occurred; the cross bars were liable to get cut away and retrogression of bed levels occurred; the canal surface slope formed a series of small cascades, and the deep pits in the bed were a source of danger to natives attempting to ford the canal. This system with the canal banks at the normal distance apart has been abandoned; it was used extensively on the Sirhind Canal. It was a mistake not to have thrown the canal banks back so as to have had wide berms, which would have given more waterway and greater inducements for silting up of the pits, while the internal silt berms would have strengthened the canal banks.

In and out system of external silting, Figure 2.—This was introduced into the Punjab from the United Provinces. Extensive reaches of many of the Punjab canals have had the banks strengthened by adoption of this

system. The disadvantages of the "in and out" system are as follows: Operations become very protracted, as the quantity of water passing in is small in amount while, if the inlets and outlets are made very wide, the canal channel becomes reduced by silting on the bed, but more especially on the berms, and heavy clearances are necessary at some time in the future. The outer banks and also the cross bunds have to be given normal strength of canal bank or costly breaches will occur. The heads of the reaches speedily silt up and their clearance is costly. Numerous costly changes of inlets outlets and cross bunds are necessary as silting up proceeds. This system has been abandoned on all new canals on account of the disadvantages just stated. The only case in existing works where it could be used with advantage would be where the width to be silted up varies from bed width to twice bed width of canal, and is too narrow to warrant the whole canal supply being turned through the reach under the system, (c), *supra*. The experience gained on the Chenab Canal was that the reaches should be from 1,000 feet to 1,500 feet in length, so as to increase the discharge and hasten silting up. Reaches on this system are requisite for completing silting up in cases where long silting reaches (c), *supra* are used. Short silting reaches of 500 feet to 800 feet in length were used on nearly every large canal in the Punjab, but in many cases those in charge got tired of the slowness of the process, of the long continued maintenance and liability to breaches, and then cut one out of every two or two out of every three cross bunds when a great improvement followed.

It may be pointed out that all spoil for banks should be got from the space to be silted up, and that outer bunds and also the cross bunds should be given the normal section of canal banks.

Long reach system, Figure 3.—This would never be used on new works since the system, to be described below, and which requires only two banks, is clearly much cheaper and less costly in maintenance. In existing canals crossing long lengths of low ground, and where the width of Government land affords space to be silted up, greater in width than about twice that of canal bed, then this system is the best to adopt. It was used on the Bari Doab, Sirhind and Chenab Canals. In the case of the Rakh Branch of the Chenab Canal the following are some of the conclusions arrived at in the case of a series of reaches, aggregating 37,000 feet in length, 185 feet in average width, and in which about 329 lakhs of cubic feet of silt were laid down.

(i) The banks of *Kalarathi* soil require a thickness of 22 feet at full supply level with 1·5 to 1 slopes, and that consolidation is satisfactorily effected by running water along the core of the bank when under construction, and that the banks should be tested for 10 to 15 days before the reaches are opened.

(ii) That masonry heads or regulators are unnecessary.

(iii) That the inlets and outlets require normal waterway of canal on a line at 45° to the canal, protection at the most exposed parts being given by stone pitching or by bushing.

(iv) That reaches of 4,000 feet to 5,000 feet work satisfactorily with a supply of 1,000 cusecs turned through.

(v) That the outer banks and cross bunds require bushing.

(vi) That open brushwood spurs are required at 100 feet intervals to guide the water in a central channel and insure rapid formation of silt berms on the sides. These spurs should be very open, and as silt soon accumulates they have little to withstand and may be made very inexpensively.

(vii) That the proportion of silt deposited to water passed through varied from 1 in 1,600 to 1 in 3,000 and was averagely 1 in 2,234.

(viii) That 3 or 4 months sufficed for one reach of 5,000 feet in length when the finishing off had to be done on the "in and out" system in reaches of 1,000 feet to 1,500 feet in length.

System of internal silting, Figure 4.—This is universally applicable in the case of all new canals and distributaries, and it is the simplest and most economical of all the systems of strengthening banks by silting which has ever been used. It was extensively used by Major J. H. Western, R.E., on the Sirhind Canal, by Mr. S. Preston on the Chenab Canal, and later by most Canal Officers in the Punjab who have been in charge of important construction works. The advantages of the system are as follows: Any desired amount of thickening of canal banks can be attained by setting them back that amount, and leaving the water to complete the operation; the extra cost involved is trifling; external borrow-pits are largely or entirely avoidable and maintenance costs very little.

The cases which arise are shown in the sketches, Figures 4 (*a*) to 4 (*d*).

In figure 4 (*a*) spoil from excavation is about equal to the bank requirements, but it is desired to have a liberal berm inside to strengthen the banks and reduce cost of maintenance. In this case we have

Width of berm=depth + an arbitrary constant.

Figure 4 (*b*) shows a case in which the depth of digging is less than balancing depth, and the extra spoil is got by excavating a widened bed, side spurs being left at 100 feet intervals projecting out to normal profile of the canal; this is applicable where the lead does not become excessive. In some cases on the Jhelum Canal the widening on each side is 30 feet to 40 feet, the canal bed being 140 feet.

Figure 4 (*c*) shows a case in which the bed is very little in soil. The spoil procurable from excavating down to a widened canal bed is short of requirements for the two banks, while outside pits are undesirable. The canal banks are thrown sufficiently far back to admit of spoil for the banks being got from inside borrow-pits; 10 feet spurs are left at 100 feet intervals to induce side silting; unexcavated 10 feet cross bunds are left opposite the spurs, and 10 feet berms are left along the toes of the banks. The widening is such that the inside excavation will furnish the requisite amount of spoil for the two banks. It is considered generally inexpedient to have pits deeper than the following:—

Main Canals of 100 feet to 250 feet bed	... ...	4 feet.
Branch do. do. 30 feet to 100 feet do.	... ...	3 „
Large distributaries of 15 feet to 30 feet bed	...	2 „

For ordinary and small distributaries, inside borrow-pits will usually be inadvisable, as the requisite spoil for the banks, if got outside from pits only 1 foot in depth, will not cause material damage to the land; however, the banks, in the case of distributaries, should be thrown back where low ground is crossed so that ultimately crest widths of 15 feet to 20 feet may be secured, which will lessen the cost of maintenance.

Figure 4 (*d*) shows a case in which canal bed is above natural surface. Spurs, side berms, and cross bars are more necessary than in the case to which figure 4 (*c*) relates; however, to obviate general deposition of silt evenly throughout the widened bed and meandering of the canal, it has been found necessary in such cases to construct, at intervals of about 150 feet, side spurs of jungle wood stakes and brushwood extending from canal banks to the normal profile of the ultimate canal channel: when this is done the side portions silt up rapidly, the deep stream retains a central position and the section required for the canal is secured.

These side spurs may consist of two lines of jungle wood stakes, 1·5 inches to 2·5 inches diameter, let into the bed about 1·5 feet; the lines of stakes may be 2 feet to 3 feet apart and the distance from stake to stake 3 feet on each line. The brushwood filling should not be packed as it is necessary that the water should flow fairly freely through the spurs. It is unnecessary that the stakes should be carried up to full supply level as canals do not carry full supply for some years after being first opened, while the side berms will continue to rise without any brushwood spurs once they attain a height of 2 feet to 3 feet over the bed of the central part of the channel.

Widening to be shown on the longitudinal sections.—Full particulars regarding widening should be shown on the longitudinal sections of channels.

PAPER No. 12.

On the Syphon proposed for passing the supply for the Lower Bari Doab Canal under the River Ravi.

The high land between the Sutlej and Ravi Rivers, south of the tract irrigated by the Upper Bari Doab Canal is known as the Montgomery Bar. Most of this Bar land, at present, lies uncultivated and is chiefly used as a camel-grazing ground. There are parallel tracts of Khadir land on either side along the Ravi and Sutlej in which there is a certain amount of scattered cultivation, and some inundation irrigation along the former river. It is proposed to irrigate these Bar and Khadir lands by a supply of water, brought from the Chenab River on the west, in a canal taking out opposite Sialkot, passing Gujranwala, and crossing the Ravi River at Mohlanwal, 21 miles south of Lahore. This canal from its offtake on the Chenab to the Ravi Crossing will irrigate the Upper Rechna Doab, and be known as the Upper Chenab; beyond the Ravi Crossing it is designated the Lower Bari Doab Canal. The reason for taking the supply of water from the Chenab River is, that the whole of the Ravi cold weather supply is abstracted by the Upper Bari Doab Canal at Madhopur, while the supply of water available in the Sutlej is proposed to be reserved for the Sutlej Valley.

The Chenab River has no surplus supply of water in the cold weather but it is proposed to bring a canal from the Jhelum River, to be known as the Upper Jhelum Canal, and taking out from a point 14 miles north-west of Jhelum and tailing into the Chenab River above Khanki Weir, where the existing Lower Chenab Canal takes off all the present cold weather supply.

There is a good supply of surplus water in the Jhelum River, and the amount of water withdrawn from the Chenab, opposite Sialkot, for the Upper Chenab and Lower Bari Doab Canals, will be compensated for by an equal amount thrown in above Khanki Weir by the Upper Jhelum Canal.

The Ravi Valley is comparatively low for many miles from the river on the west side, but on the east side the ground rises abruptly to a height of about 30 feet.

To suit the low *Khadir* land on the west of the Ravi River, the canal should be at a fairly low level so as to avoid a long heavy embankment, while on the east side a high level is desirable to lessen the amount of digging in gradually proceeding on to the crest of the ridge in the Montgomery Bar. The bed level has been fixed so as to meet these conditions and the principal levels at the crossing are as follows :—

R. L. of H. F. L. afflux in Ravi	667·1
R. L. of full supply level in canal	668·75
R. L. of canal bed	658·75
R. L. of mean bed of river	649·0
R. L. of deepest part of river bed	643·5
R. L. of spring level	653·0

The levels above given show that high flood level in the Ravi, and full supply level in the canal are nearly the same. The discharges are as follows :—

Maximum flood discharge in Ravi	200,000 cusecs.
Maximum supply in canal	6,481 „

The following methods of crossings suggested themselves :—

(i) To pass the canal over by a superpassage.

(ii) To provide a weir with under-sluices to hold up the river to the desired level, the canal being given a Head Regulator on the east side.

(*iii*) To pass the canal water under the Ravi by a steel tube syphon.

(*iv*) To adopt a masonry syphon for the canal.

These works will now be discussed.

A superpassage for the canal over the Ravi, to give a clear height of 4 feet over high flood level, would have necessited an embankment of prohibitive height for 28 miles in length for the crossing of the low Khadir tract on the west side of the Ravi. This proposal had, therefore, to be abandoned.

A weir with under-sluices would have necessitated holding up the river by from 8 feet to 16 feet. The work would be costly and would involve troublesome regulation. A considerable extent of land upstream of the work would be water-logged and there would be serious loss from percolation and leakage. These disadvantages, coupled with the fact that no reliable supply of water is procurable from the Ravi, suggested a syphon crossing.

A design for a steel tube syphon was prepared, consisting of 8 tubes of 11·4 feet diameter, which is the size given by 3 plates of 12 feet in length, but it was found—

(*i*) that the cost of a tube syphon would greatly exceed that of a masonry one, the amounts of the two estimates being Rs. 22,82,867 and Rs. 19,89,870, respectively, exclusive of training works;

(*ii*) that the steel tube syphon with $\frac{3}{8}$ inch plates, could not be regarded as a permanent work since the tubes would have to be renewed at great cost in about 25 or 30 years; renewal in the short time available would be attended with great difficulty when the canal supply had to be kept up by a temporary diversion liable to be swept away at any time in the cold weather.

The great advantage which a tube syphon has over a masonry one, is the comparatively shallow foundations required, with far less pumping than is needed for the latter. If unwatering of the deep foundations required for a masonry syphon is impossible, a tube syphon would be one of the alternatives to be considered. Meanwhile, it is proposed to rely on unwatering being found possible. It may be observed, calculations go to show, that if steel tubes 11·4 feet diameter were used they would have to be cased in concrete to obviate collapse when submitted to great pressure outside, while unwatered inside for examination or possible repairs.

Under these circumstances, a masonry syphon of a permanent character, which would not involve costly maintenance and no regulation, appeared to be the most suitable work. The great objection to a masonry syphon is the depth to which unwatering will have to be carried out, and it is necessary to adopt expedients which will reduce this depth as much as possible. The ordinary rough rule for a syphon is to make the thickness at the crown 0·4 × head on the upstream side; this rule does not represent correct theory as will be shown below, but it is approximately true and it suffices for present purposes, to observe that this rule would call for a thickness of 13·3 feet at the crown and for unwatering having to be done against 32·3 feet of water instead of 27 feet as in the masonry design prepared. The cost of this great thickness at the crown would be almost prohibitive, while it is very unlikely that unwatering in the sub-soil of sand would be possible when the head is so great as 32·3 feet.

Hence, it is proposed to adopt a syphon with the system of tie rods and steel beams shown on plans Nos. 2 and 3. It is asserted by all the advocates of reinforced concrete, that iron or steel built into masonry or concrete, and not exposed to air or running water does not oxidise; it is only necessary to rely on this as far as the straps under the inverts are concerned, as all other parts of the iron work are arranged so that they can be renewed. Even in the case of the invert straps, it will be shown that the mortar at the base of the piers will only have to withstand a force of 5 lbs. per square inch if these straps should become completely unserviceable, provided the ties up through the piers are renewed, if found necessary, and are kept perfectly serviceable. Ordinary kankar lime mortar, even if not of the best quality, can be relied on to withstand more than 5 lbs. per square inch.

The object of the inverts underneath is to distribute the pressure. The reason for having thick piers is that these may, by means of the vertical ties, assist more fully in resisting blowing up. The use of the inverts in the backing of the arches is to obviate the work failing at the crown by the upward force causing transverse stress, which concrete is quite unsuited to withstand. The brick floor on the top will admit of renewal. The karrie grooves at the crest of the entrance are to admit of regulation, so that the water may not race to the syphon, which will occur if the co-efficients have been chosen too low. The inside of the barrels is to be plastered with Portland cement to diminish the resistance, and lessen the head required. The up and downstream protection on the river bed is designed to resist severe action when the work has to act as a weir. The wing walls are given very deep foundations so as to be safe against scour outside or blowing from the great head inside. The side spurs would have been made longer but for the cost.

The two outer inverts on each side in the backing are to receive tie rods as the top of the abutments is unsuited to withstand thrust.

It will be seen that spring level is 4 feet above the top floor of the work which is at average bed level of river; having the floor at this low level greatly adds to the waterway. If the syphon were raised 7 feet, the floor being then 3 feet above mean river bed, the 8 feet thickness proposed for the crown would suffice to resist blowing up without any tie rods or steel girders and inverts in the backing, but a 8 feet weir would thus be created in the river, and to give the same water-way, the width between side walls would have to be increased from 1,300 feet to about 2,200 feet, which would add Rs. 6,00,000 to the cost; however, this is an alternative to the adoption of steel tube syphons, if unwatering to a depth of 27 feet is found by preliminary trial to be impossible. In the construction of Khanki Weir, unwatering to a depth of 22·5 feet was carried out over an area of 700 feet × 300 feet with the river water quite near.

If the syphon and river bed protection to an aggregate width of 207 feet is executed to half the length, the area to be pumped would be—

$$207 \times \frac{1{,}510}{2} = 156{,}285 \text{ square feet, say } 156{,}000 \text{ square feet,}$$

as compared with 210,000 at Khanki. The quantity of water to be pumped, if the sub-soil is the same, will probably be found proportional to—

area × square root of head,

while the work to be done would be proportional to this multiplied by the lift that is the head; hence we would have—

$$\frac{\text{Power required at Syphon}}{\text{Power used at Khanki}} = \frac{156{,}000}{210{,}000} \times \left(\frac{27}{22{\cdot}5}\right)^{\frac{3}{2}} = 0{\cdot}97.$$

The efficiency of pumps for various lifts varies, and the sub-soil may differ; the case is, therefore, one which can best be settled by trial over an experimental length of 200 to 300 feet. Experimental borings made over wide areas go to show that the foundations will be in sand, charged with water under high pressure.

Calculations for Ravi Syphon.—These are made for the masonry syphon as designed.

Afflux.—The maximum flood discharge of the river may be assumed to be 200,000 cusecs. Assuming the depth (d) to be 16′, the afflux over this 2 feet, the width (l) = 1,300 feet, that the head due to the velocity of approach may be neglected to be on the safe side, and as it is partly expended in producing the afflux, and that the co-efficient of discharge $c = 0{\cdot}8$, then the discharge D will be—

$$D = \tfrac{2}{3} c\sqrt{2g}.\, l.\, h^{\frac{3}{2}} + c\sqrt{2g}.\, l.\, d.\, h^{\frac{1}{2}}$$
$$= \tfrac{2}{3} \times 0{\cdot}8 \times 8 \times 1{,}300 \times (2)^{\frac{3}{2}} + 0{\cdot}8 \times 8 \times 1{,}300 \times 16 \times (2)^{\frac{1}{2}}$$
$$= 15{,}698 + 188{,}232 = 203{,}930.$$

This is sufficiently near to be accepted.

Discharge of Syphon.—This consists of 8 barrels of 101 square feet, hence the total area is 808 square feet. Assuming the velocity to be 8 feet per

second, the requisite head to give this velocity will be calculated for the case in which the barrels are plastered with Portland cement; and also for that in which the surface has been allowed to get worn, and may be considered as rough as that of brickwork. This question is of importance and the general theory of a syphon will now be discussed—

Let h = fall in feet through the syphon

L = length of barrels and entrances = 1,390 feet.

m = hydraulic radius of barrels

G = weight of a cubic foot of water

v = velocity

Q = discharge in cusecs

f_1 = a co-efficient such that $f_1 \frac{v^2}{2g}$ is loss of head due to friction at the entrance

f_2 = a co-efficient such that $f_2 \frac{Lv^2}{m\,2g}$ is the loss in overcoming surface resistance in the barrels

Referring to Unwin's Hydro-Mechanics, paragraphs 69, 70 and 78, we have GQh foot pounds of work expended as follows:

In imparting velocity of $\frac{GQv^2}{2g}$ foot lbs.

In overcoming entrance resistances $G\,Q\,f_1 \frac{v^2}{2g}$ foot lbs.

In overcoming surface resistance $G\,Q\,f_2 \frac{L}{m} \frac{v^2}{2g}$ foot lbs.

Equating and dividing by GQ

$$h = \left(1 + f_1 + f_2 \frac{L}{m}\right) \frac{v^2}{2g}$$

$f_1 = 0{\cdot}08$ for a bell mouth, and $0{\cdot}505$ for a cylindrical one: the latter may be accepted to be on the safe side.

$f_2 = a\left(1 + \frac{b}{m}\right)$ in which we have the following values, m the hydraulic radius being 2·7 feet:—

Cement surface $a = 0{\cdot}00294$

$b = 0{\cdot}10$

$f_2 = 0{\cdot}003$

Brick surface $a = 0{\cdot}00373$

$b = 0{\cdot}28$

$f_2 = 0{\cdot}004$

The total loss occurs as follows:—

at entrance $(1 + f_1) \frac{v^2}{2g}$

throughout length of barrels $f_2 \frac{v^2}{2g}$

Hence we have the following results—

v being $\frac{6481}{808} = 8$ feet per second, nearly, and $2g$ being 64·4.

Surface of barrels.	Entrance losses $(1+f_1)\frac{v^2}{2g}$	Loss in barrels $f_2 \frac{L}{m} \frac{v^2}{2g}$	Total loss of head h
Cement plaster	1·47	1·51	2·98 feet, say, 3·0 feet.
Brickwork	1·47	2·01	3·48 feet, say, 3·5 feet.

The head provided for is 4 feet, and will be ample. It is desirable to have surplus head as the loss by the water being turned through four right angles has been ignored, but so has the velocity of approach, which partly compensates.

The head under the soffit of the arches will gradually increase from the downstream end to the upstream end, the increase being 1·51 feet for cement and 2·01 feet for brickwork; immediately at the upper end there will be a further sudden increase of 1·47 feet.

The head over soffit at downstream end will be 23·75 feet.

„ „ upsteam „ 25·26 feet for cement.

„ „ „ „ 25·76 feet for brickwork.

The work will be safe if it can withstand a head of 25·76 feet, say 26 feet throughout its length.

Flotation.—Considering a length of one foot of one barrel when the work is unwatered—both externally and internally, the weight of the masonry (assumed averagely as 112 lbs. per cubic foot) should suffice to obviate flotation by the sub-soil pressure on the foundation due to a head of 27 feet of water.

The weight of the masonry is (14′ × 23′—101) 112=24,752 lbs. and the pressure of the water is 14′×27′×62·4 lbs.=23,587 lbs. The former exceeds the latter by 1,165 lbs. per foot run of each arch.

In practice the work should not be unwatered on the top at the same time, as this is done internally, but even if carried out, it will be safe.

Blowing up.—This cannot occur by fracture at the crown owing to the arches in the backing; it is necessary to show that the load over springing of arches R. L. 636·5 and over springing of the lower inverts R. L. 832 is sufficient.

The upward pressure of the water on the soffit of the arch is—

26·67′×10′×62·4 lbs.=16,642 lbs. (*i*)

The forces acting downwards are as follows:—

Weight of water over floor R. L. 649·0 to 653·0 (14′×4′×62·4) =3,494 lbs. (*ii*)

Weight of masonry R. L. 636·5 to 649·0 (14′×10′+4′×1·5′—$\frac{2}{3}$×2′×10′) 112=14862 lbs. (*iii*)

Weight of pier R. L. 629·5 to 636·5 (5·5′×4′×112)=2,464 lbs. (*iv*)

As the top floor might be unwatered, when full-supply was being run in the canal, the item (*ii*) must be left out of consideration.

Item (*iii*), being less than (*i*), shows that the arch must be tied down. Items (*iii*) and (*iv*) amount to 17,326 and exceed (*i*) by 680 lbs. This is a very small margin, but no account has been taken of—

(*a*) the tie bars under invert,

(*b*) adhesion of the mortar at the bottom of the pier which at even 5 lbs. per square inch, would amount to 2,800 lbs. per foot run of each barrel.

It is very unlikely, but possible, that a flood of 1 foot or even 2 feet might accidentally pass into the canal, and, if so, item (*i*) would become 18,387 lbs.

It will be seen that this exceeds the sum of items (*iii*) and (*iv*) and either we must rely on the adhesion of the mortar at the base of the pier or on the straps under the invert. The invert straps may be put in to make the work perfectly secure in the early stages, but it is clear that when the mortar has set fully, reliance can be placed on it, and the invert straps need not be renewed.

Ties through piers.—The maximum force on these per foot run will be as follows:—

	lbs.
Pier R. L. 632·0 to 638·0	=2,688
Add difference 18,387—17,326	=1,061
	3,749

This for a length of 5 feet from tie to tie amounts to 8·4 tons. The tie rods, with a net area of 3·5 square inches, suffice, and make ample allowance for rust reducing the effective area.

Steel channel irons.—The load on these would be 8·4 tons, uniformly distributed over the twin section on spans of 5 feet. As one channel iron would bear 8·1 tons, there is clearly ample strength as well as allowance for deterioration.

Tie rods for channel irons.—It is rather uncertain what this will be as the tension will depend on the adhesion between the invert in backing and the concrete. The total upward vertical load will differ little from 3,749 lbs. per foot run of arch, say 1·7 tons, *vide supra.* The versine being 3 feet to bottom of middle third of arch, and the ties 2·5 feet apart, the tension on each will be $T = \frac{1 \cdot 7}{2} \times \frac{14}{4} \times 2 \cdot 5 \times \frac{1}{3} = 2 \cdot 5$ tons.

The tie rods $1\frac{1}{4}$ inches diameter, have an area of 1·25 square inches, which will withstand over 5 tons, and provides amply for reduction at ends as well as for deterioration.

Invert straps.—These will have to bear a force of 1,061 lbs. per foot run of arch, that is 5,305 lbs. per bar, or 2·5 tons. Single bars of 1·75 square inches net area at the joints will have ample strength.

Arches.—The factor of safety for the brick arches is about 8 and detailed calculations appear uncalled for here.

PAPER No. 13.

The Thapangaing Aqueduct at 6½ miles on the Mandalay Canal.

The Thapangaing Nullah is the largest and most important of the drainage lines that cross the Mandalay Canal. It may be described as a hill torrent with an estimated catchment area of 171·64 square miles. Though the stream is perennial, its cold weather discharge is insignificant, about 20 cusecs; but during the monsoon season (May to November) it is subject to heavy floods which rise and fall with great rapidity.

The maximum flood discharge was for long a matter of conjecture; the rapid rises and subsidences, and the character of the nullah itself, rendered accurate observations extremely difficult. The torrent bed varies in width from 20 to 30 feet, and follows a meandering course in a valley densely covered with *kine* grass 10 to 18 feet high, shrubs, creepers, trees and luxuriant vegetation of all kinds, so that the channel does not lend itself to very accurate discharge measurements.

In the matter of design the Aqueduct to carry the Mandalay Canal across the Thapangaing valley passed through various phases. The original project estimate provided three spans of 30 feet, with a waterway of 870 square feet, to pass an estimated discharge of 5,347 cusecs, with a velocity of 6·1 feet per second.

Later calculations placed the flood volume at 17,760 cusecs, to provide for which a second design, with 6 spans of 30 feet, and 2,220 square foot waterway was prepared. But even the adequacy of this provision was doubtful, and further calculations based on the flood marks of 1894, pointed to a discharge of something approaching 40,000 cusecs. The Inspector-General of Irrigation, on his inspection of 1896, doubted the correctness of these calculations, and directed the preparation of a design with 12 spans of 21 feet and a waterway of 3,000 square feet, to provide for a discharge of 24,000 cusecs and a velocity of 8 feet per second. The design based on these orders provided 13 spans of 22 feet, a waterway of 2,926 square feet, and assumed a discharge of 24,110 cusecs. With certain modifications, and a reduction of the number of spans from 13 to 12, this estimate amounting to Rs. 2,13,545 was sanctioned and the work started.

Commenced in 1898, the work progressed with many difficulties in the way of floods, and sickness, especially the cholera outbreak of 1899, so that by November of that year only 4 bays were up to springing level of arches, while the other 8 were in the foundation stage. On the 1st November 1899, occurred the maximum flood on record, the Thapangaing river rose 20 feet between 7 A.M. and noon, and the calculations indicated a discharge of 56,273 cusecs. This figure was so much in excess of that allowed in the project that the design had to be entirely reconsidered. It was ultimately decided to adhere to the sanctioned plan as far as possible, but to substitute wooden shutters for the masonry parapet walls so that a discharge of 60,000 cusecs would pass partly under and partly over the work. This final design of the aqueduct is exhibited by the drawings. The work was re-commenced in December 1899 and completed by the close of 1901 at a cost of Rs. 3,84,471.

The design of the work is sufficiently indicated by the accompanying plan, Plate 25, and it is only necessary to explain the wooden shutters. The length 300 feet of Aqueduct is closed by 60 pairs of shutters, spaced 46 feet apart for the canal waterway, each shutter being 4 feet 11¾ inches long by 7 feet high, except the pair on south side, which is one foot shorter. Each pair of shutters is connected at the upper end by a wire rope attached to the triangular brackets fixed on the top of the shutter, and is pivoted at lower end. The pivots consist of cast-iron trunnions secured to the shutter by an iron strap, which is bolted along its whole length. The trunnions rest on saddles secured by straps to short-rolled beams built vertically into the masonry exactly below the ends of the shutters. The shutters are also held erect and prevented from falling

towards the canal by tension rods, fastened at their upper end to the brackets on the shutter; the lower end of the up-stream tension rod engages by a hook with the let-go gearing. Leakage between and below shutters is prevented by a staunching arrangement of one inch strips of rubber which are pressed over the vertical joint on the canal side by strips of wood fixed to shutter by small springs and screws, and on lower end of shutter by hoop iron. The end of this rubber strip at lower end of shutter presses against a similar 1 inch strip of rubber which is supported horizontally outside the shutter the whole length of Aqueduct, and closes a $\frac{3}{4}$ inch gap between the angle iron fixed along the bottom of the shutter and a bar of iron built into the masonry sill parallel to the angle iron.

The let-go gear is built into the masonry sill opposite the centre of each up-stream shutter, and consists of a lever and trigger arrangement, of which the lever is actuated by one of a series of lead balls fastened on a wire rope, while the trigger secures the hooked end of the upper tension rod. The wire rope of the let-go gear winds on to the drum of a winch fastened down on the top of the wing wall, and passing over two pulleys traverses the whole length up-stream of the shutters, and over two pulleys, and ends in a suspended counter-weight. Lead balls are fastened at intervals on the wire rope to actuate the let-go gear.

There is telephonic communication between the Aqueduct and head works, as well as down the canal. In the event of a rising flood the canal head is closed as soon as a certain level is reached, the needles are put into the regulator below the Aqueduct, and when the flood has reached another fixed level the winch is manned and as many shutters as are considered necessary are lowered. Only on two occasions in 1902 were shutters lowered successfully without damage, all other floods passing below through the tunnel way.

PAPER No 14.

On Steel Tube Syphons on the Kalpani Distributary, Swat River Canal.

The Kalpani Distributary of the Swat River Canal is a fair-sized channel with a designed discharge of 94 cusecs. It has a bed width of 8·0 feet, a full supply depth of 5·0 feet and a bed slope of 1 in 4,000.

The alignment of the distributary, like that of the main canal itself, follows roughly a contour of the country, and in consequence crosses the natural drainage lines almost at right angles. The largest of these drainages are the Bhagiari and Kalpani torrents or nullahs, which are crossed by means of steel tube syphons, which form the subject of this Paper.

The Bhagiari is a tributary of, though not much smaller than, the Kalpani. They are about a mile apart where the distributary alignment crosses them, and meet about 2½ miles lower down. They rise on the southern slopes of the mountain range which shuts out the Swat and Boner countries, situated about 20 miles to the north in a direct line, and they drain a combined area of about 400 square miles before they effect a junction. The Bhagiari is the smaller of the two, and rises on the west, near Malakand, while the Kalpani comes down more directly from the north, from the direction of the Morah pass. The former nullah is dry for the greater part of the year, but the Kalpani generally has some water flowing down it except at extremely dry periods. The channels are very deep in places and tortuous, but with steep bed slopes, and where the distributary crosses them they are from 200 feet to 300 feet wide, from 20 feet to 25 feet below the level of the country, and have slopes of about 7½ feet per mile.

No measurements of flood discharges have been made, but after heavy rainfall in the mountains it is quite conceivable, judging from the catchment areas, that these nullahs can discharge 30,000 and 40,000 cusecs, respectively, and they have been known to run with depths of 20 feet or so.

Now it is evident that brick or stone aqueducts would have been very expensive, if employed to convey the water of the distributary across these nullahs, besides being exposed to danger from violent floods, the volumes of which are uncertain. On the other hand, these tube syphons, by the sacrifice of a certain amount of "head" or command in the distributary, afforded a cheap and safe means of crossing. The loss of command is not very great, however, owing to the rapid slope of the country.

Owing to the difference in cross section at sites, the Bhagiari tube (for the smaller channel) is 305 feet long, while the Kalpani tube is 246 feet in length. The latter again has more bends in it, and is not symmetrical with regard to the centre line of the nullah. The Bhagiari tube has been selected as a type and is described in detail below.

Plate No. 27 shows a longitudinal section of the tube with its brickwork approach and exit chambers. The tube is formed of $\frac{3}{16}$ inch mild steel plates, 12 feet long and 3 feet 10 inches wide, bent round in the direction of the length and lap-jointed. The 12-foot plate after allowing for the lap-joint gives an internal diameter of 3·75 feet, and each section of the tube thus formed may conveniently be styled a plate-unit.

For facility of transport and erection seven such plate-units were lap-jointed and rivetted together to form a pipe-length of 25 feet before being sent out of the shops. The ends of these pipe-lengths are provided with planed angle iron flanges, and with bolts and nuts for fastening them together at site of the work.

The joints will be described in fuller detail below. To proceed with the general description of the tube. This is cased in concrete with a minimum thickness of 18 inches as shown in the cross-section, partly to protect it from accidental damage by scour, but chiefly to prevent corrosion by excluding the

air, and the deteriorating influence of alternate wetness and dryness. The inside of the tube was coated with bituminous paint with the same object; but with silt-bearing water flowing through at a velocity of 8 feet or 9 feet per second it is highly improbable that the paint or varnish lasted any time.

The tube is made of sufficient length to place the foundations of the brick-work approach and exit chambers well in soil, and is given a slope of one foot down to the centre of the nullah from each side, to facilitate unwatering when found necessary through the escape valve which is placed there. A detail plan of this valve is shown, Fig. 2.

Plate No. 27 also gives the sections of the brickwork approach and exit chambers (or upstream and downstream wells as they may be called for short) and shows the conoidal shape of the entry and exit at junction of the tube with the well. This portion of the brickwork was laid in Portland cement.

The area of these wells is 36 square feet as compared with 11·04 of the tube. Figure 3 is a plan of the downstream well. The upstream well is similar except that it is provided with grooves for planks, as shown in dotted lines, in case the friction head in practice should prove to be much less than that allowed. In this case planks could be dropped and *racing* in the upstream channel prevented. Iron gratings are provided on the upstream entrance to prevent jungle drift getting into the tube, and iron railings are also fixed round the wells themselves.

The vertical joints connecting the plate-units together have 2-inch laps and are single-rivetted with $\frac{1}{2}$-inch rivets at $1\frac{5}{8}$ inch pitch. The vertical joins of the 25-foot pipe-lengths are flanged, as already stated, and are bolted together with bolts and nuts. The angle iron of the flanges is $3 \times 3 \times \frac{1}{2}$, and has planed faces. A piece of tarred felt was placed between the planed faces of the adjoining pipe-lengths before they were bolted together.

The horizontal joints of the plate-units are also lapped but double rivetted with 2-inch pitch, and the joints of alternate units are placed on opposite sides.

The pipe-lengths were all tested to 60 lbs. pressure before being sent out of the shops. The maximum pressure head on the tube is about 40 feet, equivalent to a pressure of $(0·434 \times 40) = 17·36$ lbs. per square inch; while the tension produced is $(17·36 \times 22·5) = 390$ lbs. per square inch, for which a shell ·07 inch thick would suffice, with a factor of safety of six, as compared with 0·1875 inch ($\frac{3}{16}$ inch) provided. The thickness is of course determined by other considerations, such as risk of damage in handling and carriage and corrosion, besides those of the resistance necessary to counteract the bursting pressure.

To allow for expansion and contraction due to thermal changes four expansion joints have been fixed as shown, one in each of the sloping sides and two on the level portion of the tube. And, in order that movement of the tube may not be prevented by the flange connections of the 25-foot pipe-lengths, the flanges have been built round with brickwork, space being left all round the angles for clay filling Figure 4, shows this arrangement in detail. All rivet lines were also embedded in clay puddle with the same object. Figure No. 5 gives the detail of the expansion joint. It consists of a flat iron annulus 7·0 feet in diameter, connected to the angles of the pipe-length by thin steel wings, thus forming a gap 4 inches wide. It is designed in fact to work like the bellows of a concertina.

With a variation of 30° F. in the temperature it was calculated that the expansion for the 300 feet length of tube would amount to about $\frac{3}{4}$ inch, thus:—

Expansion for 180° F. = ·0012 of length,
Ditto for 30° F. = ·0002 of length,
and for 300 feet of tube = ·06 feet = 0·72 inch,

say $\frac{3}{4}$ of an inch.

These syphons were built in 1897-98, and cost excluding percentages for Establishment and Tools and Plant—

	Rs.
Bhagiari Syphon	27,565
Kalpani Syphon	28,330

or Rs. 90·4 and Rs. 115 per foot run of tube, respectively. The difference in cost is due to the difficulty experienced at the Kalpani site with water in the foundations, necessitating more timbering and heavier pumping.

As the bed slopes of the nullah are steep, some of the unwatering was done by means of drains and the rest by pumping.

When these syphons were first brought into operation it was found that water crept along the side walls and breached out, but the well puddled banks soon tamped themselves, and breaching gave comparatively little trouble.

An abstract of the quantities and cost by sub-heads is given in the appendix as they may be of interest.

There is a fall through the syphon of 4·0 feet from a water surface at R. L. 1,004·63 on the upstream side to R. L. 1,000·63 on the downstream side (Plate No. 27), the formula used being as under:—

$$V = 8{\cdot}025\sqrt{\frac{hd}{(1+f_o)\,d + 4fl}}$$

where h = the head (assumed to be 4·0 feet)

d = diameter of tube

l = length of tube = 305 feet

f_o = 0·505

f = ·00511

The area of the tube is 11·04 square feet, and the velocity necessary to pass the full supply discharge of 94 cusecs is $8\frac{1}{2}$ feet per second. The value of V is found from the above equation is 9·15, thus giving a slight margin.

Observation has shown that the "head" necessary to pass the discharges is a good deal less than that allowed originally. These experimental results, are given in the following paragraph.

The results of four carefully made experiments on the discharges passing through with their resulting heads are given in the tabular statement below It may be mentioned that the co-efficient of discharge for the syphon as a whole, that is the value of c in the formula—

$$V = c\sqrt{2gh}$$

is 0·67.

Table No. I showing results of observations made in 1903.

Discharge in cusecs.	Observed head.	Velocity through tube.	Head due to velocity and entry ($1{\cdot}08\ v^2 \div 2g$).	Head due to bends (Weisbach's formula).	Difference head due to friction.	Deduced value of f in formula $hf = \frac{v^2}{2g} \times \frac{4fl}{d}$
83·5	1·96	7·562	0·9591	0·0206	0·9713	0·00336
87·5	2·18	7·924	1·0532	0·0326	1·0942	0·00345
92·0	2·40	8·332	1·1642	0·0360	1·1998	0·00342
102·0	2·79	9·237	1·4311	0·0442	1·3147	0·00305

The first two columns give the observed discharge and head. Column 3 is the velocity deduced from the area of the tube. Column 4 is the calculated head due to the velocity and the resistance at entry, the co-efficient for a conoidal entry having been taken at ·08,

Column 5, head due to the bends, has been calculated from Weisbach's formula—

$$F = \cdot946 \left(\operatorname{Sin}\frac{\theta}{2}\right)^2 + 2\cdot05 \left(\operatorname{Sin}\frac{\theta}{2}\right)^4$$

There are two bends (knees) of 15° each.

Column 6 shows the difference (or the remaining head) between column 2 and the sum of columns 4 and 5.

This must represent the head due to the friction. Applying this friction head due so deduced in the ordinary formula—

$$hf = \frac{v^2}{2g} \times \frac{4fl}{d}$$

the values of the friction co-efficient have been calculated and shown in column 7. The average of the four experiments is—

$$f = \cdot00332$$

This is much lower than the values given for rivetted pipes by Merriman (page 209), where the friction *factor* (or $4f$) is entered.

A test made with the exponential formula propounded by Claxton Fidler, for "bare metal pipes, rivetted joints" gives very close results, especially for the first three experiments, and seems to establish its applicability to similar cases. The formula is—

$$S = \frac{h}{l} = 0\cdot0000871 \; \frac{v^{1\cdot77}}{r^{1\cdot18}}$$

and the results of its application are compared below with the deduced friction heads in column 6 of Table No. I above.

Table No. II.

Serial No.	Deduced friction head from column 6 of table above.	Friction head as calculated by exponential formula.	Difference.	REMARKS.
1	0·3713	1·0294	·0581	Greatest difference in No 4 as shown also by the deduced values of f in column 7 of Table No. I.
2	1·0942	1·1181	·0239	
3	1·1998	1·2221	·0 223	
4	1·3147	1·4667	·1520	

It was not a very simple matter measuring the heads owing to the agitated condition of the entering and issuing stream, but the results of the observation are consistent and reliable.

It is only necessary to make a few remarks on the expansion joints and the concrete casing. If the modulus of elasticity of the steel be taken at 12,000 tons per square inch, a variation of 30° F. in the temperature will produce a stress of 2·4 tons per square inch in the metal. The section of the metal is $(144 \times \frac{3}{16}) = 27$ square inches, so that the total thrust or tension produced, as the case may be, is 63 tons, nearly. To avoid this stress the tube must either slide inside the concrete casing as in a sleeve (in which case the concrete would not be an efficient protection against corrosion), or else the tube and its concrete casing must move together. This latter alternative is almost impossible. The surface area of the concrete casing is$(27 \times 170 =)$ 4,590 square feet taking the straight portion of the tube only, so that a frictional resistance of only 32 lbs. per square foot would suffice prevent any movement.

On the other hand, if the tube is to slide inside the concrete casing, the adhesive strength of the mortar in the concrete must be overcome. The area of the outer surface of the tube is $(12 \times 170 =)$ 2,040 square feet, or 293,760 square inches. An adhesive strength of half a pound per square inch would equalize the stress of 65 tons, and it would have to be very inferior mortar that would not exert an adhesive force twenty times as great as this,

except when fresh. It appears then that the expansion joints are not of any use, unless the difference in the rates of expansion of the steel and its casing causes cracks in the latter.

Rankine does not give the amount of the expansion of concrete, but in Hurst's pocket-book it is given as 1 part in 700 for Portland cement and gravel, $=\cdot001428$ of the length for 180° F., somewhat greater than the linear expansion of steel which is ·0012 of the length. The probability is that the expansion joints are of no use.

The value of a good concrete casing is undoubted. It is expensive, but if well laid on should prevent corrosion of the outer surface of the tube. The inner surface must wear away gradually under the action of silt-bearing water moving at a high velocity, but the shell can wear down very thin and still exert sufficient resistance to the bursting pressure, to 0·07 inch with factor of safety of six. What the life of such a tube is, is not known, but it must be reckoned in years, and by the time the tube wears down, the concrete should have set with sufficient strength to withstand the pressure unsupported. For example, in the present case the upward pressure against the tube would be 9,375 lbs. per foot in length. The resisting area is $(2\times1\cdot5\times12\times12)=432$ square inches, and would have to bear a tensile stress of 22 lbs. per square inch, neglecting the earth covering over the concrete casing. The concrete would of course not stand the rubbing of the silt-laden water moving at a high velocity without constant attention, but even if it did not answer as a tube alone it probably doubles the life of the steel tube encased in it by the protection afforded to the outer surface.

APPENDIX.

Details of cost of Tube Syphons by sub-heads.

Bhagiari Syphon.			Kalpani Syphon.		
Sub-head.	Quantity.	Cost.	Sub-head.	Quantity.	Cost.
		Rs.			Rs.
Earthwork, dry	175,200 c. ft.	924	Earthwork excavation, dry.	361,914 c. ft.	1,551
Earthwork, wet	19,500 ,,	233	Earthwork excavation, wet.	61,346 ,,	490
Concrete	13,012 ,,	2,947	Concrete	13,951 ,,	2,686
Brickwork in Portland cement.	6,993 ,,	3,640	Brickwork in Portland cement.	6,796 ,,	3,852
Brickwork	12,217 ,,	4,069	Brickwork	13,445 ,,	4,157
Stone-pitching	12,940 ,,	1,914	Stone pitching	11,176 ,,	952
Woodwork	750 ,,	949	Woodwork	700 ,,	1,010
Standpipe	1 No.	20	Mild steel tubing	257·5 l. ft.	7,976
Mild steel tubing	305 l. ft.	9,202	Pumping	...	4,455
Expansion joints	4 No.	598	Standpipe	1 No.	19
Pumping	...	2,020	Expansion joints	3 ,,	440
Puddle	6,608 c. ft.	139	Puddle	15,824 c. ft.	363
Contingencies	...	910	Cement plaster	...	52
			Contingencies	...	327
Total	...	27,565	Total	...	28,330

PAPER No. 15.

On the Design of the Suketar Superpassage, Upper Jhelum Canal Project.

The Suketar Torrent is a large torrent that rises on the southern slopes of the great Pir Panjal range of mountains, and flows into the river Jhelum, on the left bank, about 8 miles in a direct line above the town of that name. The Upper Jhelum Canal alignment crosses the two main branches of the torrent which join about two miles further down, at a spot only a few miles above the junction of the combined streams with the river Jhelum.

There is generally a small stream flowing, down the larger of the two branches except in the very dry season.

Two superpassages had accordingly to be designed, one at R. D. 77,000 of the alignment, for the main branch, and the other at R. D. 90,000 for the smaller branch.

Superpassage No. I is designed to pass 52,584 cusecs, and superpassage No. II just about one-third this volume or 17,510 cusecs.

It is unnecessary to detail the peculiar conditions which necessitated the designing of two superpassages in place of one. It will suffice to describe superpassage No. I, which is a type for the other.

The canal here is of great size. The maximum discharge is taken to be 8,500 cusecs, with a bed width of 220 feet and a full supply depth of 9·5 feet. The bed slope of the canal has been fixed at 1 in 6,666 or 0·15 per 1,000.

The relative levels are—

Canal bed	808·45
Canal bank	821·00
Torrent bed	805·00
Superpassage floor	808·00

It will be seen that the torrent bed is humped 3·0 feet at the superpassage, a rapid being given on the downstream side to get down to the bed of the stream again. The object of this was to maintain a free outfall for the superpassage in case of silting of the torrent bed, and to reduce the thickness of the superpassage floor. Experience on the Superpassages of the Sirhind Canal pointed to the necessity for attention to the downstream conditions as affecting the discharging capacity of the superpassage. This humping is therefore one of the features of the design.

In superpassage No. I the width between parapet walls is 454 feet, and in superpassage No. II the width is 152 feet. With these widths the flood discharges are calculated to be passed with a depth of 9·5 feet over floor, and a heading up of 2·5 feet on the upstream side. Section B B of the accompanying plan (Plate No. 20) shows the stream line over the superpassage and down the rapid of 1 in 15 on the downstream side.

The parapet walls are designed to be 3·9 feet above highest flood level including afflux at the upstream end, and 3·6 feet above water level over superpassage at the downstream end.

To maintain the requisite velocity of about 12 feet per second across the superpassage, the floor has been given a slope of 1 in 900 as shown on section B B.

As will be seen from the plan the upstream wings splay out much more rapidly than those downstream. At the upper end, where the torrent bed is ramped up to the superpassage floor, a 2·0 feet thickness of pitching over 0·4 feet of quarry chips has been provided. At the lower end is the rapid of 1 in 15 slope with an aggregate depth of 3·5 feet of protection in hydraulic mortar, and a talus beyond that of concrete blocks, 4′ × 4′ × 2′ and 100 feet in length. Beyond the concrete blocks again 20 feet of dry stone pitching 4·0 feet in depth has been given to adjust itself to the scour.

The full supply discharge of the canal as already mentioned is 8,500 cusecs, with a depth of 9·5 feet. This will be passed through 12 vents or openings of 15 feet span each, at an estimated velocity of 5 feet per second. As a safe allowance for the loss of head by friction in passing through the long barrels, the canal bed downstream of the superpassage has been placed 2·0 feet below the upstream level. A raised sill will be given on the upstream side to prevent acceleration of velocity, if necessary.

The thickness of the floor of the superpassage (required to withstand the blowing-up pressure) has been carefully considered. In the case of the upper superpassage, it has been made 6·8 feet and in the lower 6·5 feet.

The barrels are provided with inverts 2·1 feet thick, and a dip of one-tenth of the span, so as to distribute the pressure. Attention has also been paid to the necessity for prolonging the inverts outside the line of the face-wall of the arches so as to distribute the pressure, and to obviate the risk of failure in the foundations. The section A A shows the prolongations of the piers and inverts; and also the bell-mouthed shape of the arches, and the offset of 1·0 ft. at the foot of the parapet walls. A ramp of 1 in 3 is given up to the bed of canal on the downstream side.

The intensity of pressure on the foundations is estimated to be 2,620 lbs per square foot under the parapet walls, and 3,315 lbs. per square foot under the body of the superpassage. These figures are based on the assumption that the extreme limit of pressure coincides with a line drawn at an angle of 45° from pier base. By prolonging the inverts beyond the face-walls the pressure intensity under the parapets works out to less than that under the mass of the superpassage. It is believed that with these moderate intensities of pressure the work will be stable.

The wing and parapet sections do not call for any special remarks. The flanks upstream and downstream will be connected with the high ground on either side by high and strong embankments, with the water slopes pitched for certain distances.

The upper superpassage is estimated to cost Rs. 9,60,867 and the lower one Rs. 4,77,658, the rates per cusec of maximum discharge working out to Rs. 18·3 and Rs. 27·2, respectively.

PAPER No. 16.

Brief Notes on Absorption Losses on Canals, etc.

The simplest method of recording these losses is by so many cusecs per million square feet of surface area of the water. It is a needless refinement to take wetted perimeters instead of the surface; true, the actual seepage into the soil will be more nearly in proportion to the former, but the evaporation, which is included in all such data, varies as the latter. Besides, the loss is not by any means proportionate to the wetted perimeter, as the depth comes into the equation in an unknown degree. It is, of course, useless to gauge the loss as a percentage of the discharge, as the same channel section, with varying velocities, will have the same loss but different discharges.

All such losses vary immensely with the kind of soil, with the time the supply has been running, and the size and section of channel, etc., so that the figures given in the attached statement are true in a general sense only, but I have, at the same time, distinguished between those which are based on actual experiment, and those deduced from experience as being probable; these latter being in brackets. The data shown are for two canals, the Bari Doab Canal and the Sirhind Canal; the former being typical for good loamy soils, and the latter shewing the other extreme of poor sandy soil. It will be noticed that the losses in the latter are from 2½ to 3 times as much as the former under similar conditions.

When the supply is first run on, the loss is, of course, at its highest and it then decreases very gradually as time goes on, reaching eventually a limit. In loamy soil it was found that on a field the loss on the 8th day was 0·56 of that on the first day, the area being kept constantly flooded (see Irrigation Branch Paper No. 10). The same principle will hold good for channels alternately closed and open, as in the case of Branch *tatils*, so that the excess loss on such channels when newly opened will partially neutralise the economy from distributing by *tatils*. There are no data to determine exactly how much we lose this way, but it will certainly not be anything like so much as that required to counterbalance the advantages derived from working Branches alternately.

It may be noted that the result of the experiments on the Bari Doab Canal, taken in 1883, went to show that out of every 100 cusecs entering the canal in the winter we lost, in Main Line and Branches, 20 cusecs; in Distributaries, 6 cusecs; in water-courses, 21 cusecs; and by waste at the fields in various ways, 25 cusecs: so that the remaining 28 cusecs could have done all the useful work. Since 1883, however, we have somewhat reduced the last two items of loss; how much, however, cannot be known;—possibly our "efficiency" may now be 35 per cent. at the outside, instead of 28 per cent. On the Sirhind Canal it must be much worse than this, the soil being more porous, possibly as low as 25 per cent.

Absorption Loss Data.

Particulars.		Loss in cusecs per million square feet of water surface. Minimum.	Maximum.	Average.
	ON BARI DOAB CANAL—GOOD SOIL GENERALLY.			
Main Line .	Discharge about 4,000 cusecs, depth about 6 feet, all in shingle and sandy soil.	...	...	9·7
Branches .	Discharges from 1,000 to 3,000 cusecs, soil good loam not sandy, with silted berms, but no fine silt on bed.	...	...	2·2
Distributaries .	Discharges 30 to 100 cusecs, good loam soil, silted side berms and in lower reaches, fine silt on bed.	2·3	4·4	3·3
Water-courses .	Discharges 0·50 to 3·0 cusecs, good loam soil generally rough bed and banks, in all sorts of conditions, some new.	3·3	30·0	9·4
On fields . .	When first water is laid on, sometimes soil was moist and in some cases quite dry.	5·5	16·0	8·0
	ON SIRHIND CANAL—SANDY SOIL GENERALLY.			
Main Line (first 26 miles).	Discharge about 4,000 cusecs, depth about 7 feet, all in sandy soil, no shingle, sand silted bed and no side berms. The subsoil water table was close to surface and sloping away to the river from the canal at a grade of 2·4 feet in 1,000.	...	...	9·0
Branches . .	Discharges from 2,000 to 4,000 cusecs, depth about 7 feet, all sandy soil, little or no side berms, and sand on bed.	...	...	5·2
Distributaries .	Discharges 30 to 100 cusecs, all sandy soil, in all sorts of conditions as to bed and berms.	(5·0)	(12·0)	(8·0)
Water-courses .	Discharges 0·50 to 3·0 cusecs, all sandy soil in all sorts of repair.	(7·0)	(60·0)	(22·0)
On fields . .	When water is first laid on . . .	(12·0)	40·0	(21·0)

PAPER No. 17.

Earthen Dams.

The author had recently to examine the plans and estimates for a large earthen dam at Maladevi it was proposed to construct on bad foundations and believing that the bank would not be safe, gave his reason at length in a report which the Bombay Government printed. Extracts from this report are given below:—The Bombay Government called for actual cross sections of a number of tank embankments and for experiments to be made in a few cases to determine the saturation of the bank. These experiments are not complete but one or two results have been received and are given.

The additional information thus obtained confirms the author in the opinion given in paragraphs (*a*), (*b*), (*c*) and (*d*) below:—

(*a*) The cross section of the bank should be proportionate to the depth of water and the slope from the point where the full supply level touches the bank to the outer toe should be not less than 4 to 1 for banks of ordinary construction in less than 40 feet depth of water. This line is the hydraulic gradient for the bank.

(*b*) The earth in the bank will become saturated in time and in high banks the lower part will move and slip under the pressure of the upper portion and water, unless held down by some material not affected by water.

(*c*) In the old design a moorum casing was specified to keep the interior from cracking but the casing in most of our banks is not thick enough and in future designs should be increased to give a mass of sound drainage material which will weight down the rear slope of the bank, the thickness of the drainage material to increase with the height.

(*d*) For depths of water greater than 40 feet the behaviour of the bank when saturated is uncertain, and the cross section must be greater than for a bank of less depth.

Up to 40 feet depth the section recommended by the author has a core of selected watertight earth 10 feet wide at Highest Flood Level, and with side slopes $1\frac{1}{2}$ to 1 on both sides. This core is to be protected on the top and water side by material not likely to slip when wet, like the soft moorum or decomposed rock of the Deccan, and the slope on the water face may be $2\frac{1}{2}$ to 1 or 3 to 1.

The watertight core should be covered on the rear slope by a mass of material not affected by water and, to keep the earth from being forced into the drainage material, it should be arranged like a filter, with soft moorum on the inside against the earth core and large coarse material on the outer side, broken metal or screened gravel being very suitable for the outside.

At the rear toe provision must be made for the water to escape, and the toe must be a mass of dry stone when the foundations are good.

The best foundation for the rear toe of a bank is porous rock like moorum, or rock, and the whole of the rear mass of drainage should be carried down to such rock when it is within reasonable depth.

When no such moorum or rock is available then the site is not desirable, but if a bank has to be built a large trench should be excavated beneath the rear toe and filled with good drainage material and a berm formed over it.

As all banks get thoroughly saturated and as all earth swells when wetted, it seems to the author that it is not desirable to water the bank during construction but that it is better to lay the material dry; it is no use watering the

moorum and drainage material, and the earthen core should also be laid dry but the material must be thoroughly broken to not larger than ½ inch cubes.

Carts can nearly always be obtained, and if all the earth be carted on to the bank and distributed from the carts when in motion and if a heavy beam be drawn over the material so spread out to pulverise it a very excellent bank is formed.

A few carts lightly loaded can be used to give additional consolidation and are better than rollers.

Before using any earth for the core of the bank it is desirable to test it by making a small tank of the earth, filling the tank with water and so saturating the earth and then to allow the bank to dry. If the earth then cracks largely it is not suitable to use plain but must be mixed with moorum or some similar material to prevent cracking.

If a masonry core wall be used in an earthen dam it must be supported by earth backing to enable it to resist the thrust of the saturated material on the upstream side of the wall.

Masonry and concrete are not impervious to water, and moisture will pass through and reach the downstream material so that if this material were ordinary earth it must at least have a slope somewhat in excess or ordinary road embankments.

The cost of masonry or concrete core wall will be at least eight times the cost of a similar quantity of earth bank, if the core wall were 10 feet thick at 40 feet depth and 2 feet thick at the top, then the extra cost would be equal to 70 feet width of embankment at the bottom and to 14 feet width at the top; this is equal to a wedge shaped mass on the rear slope tapering from zero at the top to 84 feet width at 40 feet depth. It seems doubtful whether 10 feet thickness of masonry core wall is as good as 84 feet extra width of backing of drainage material to the watertight core of an ordinary bank.

Extract from Mr. Hill's note on Maladevi Earthen Dam.

A great deal of experience has in recent years been gained of the behaviour of earth in high earthen dams.

Waghad, Nehr, Mhasvad, Ekruk and Ashti have all slipped seriously; so much so that the Government of India withdrew their sanction for the construction of a high earth dam at Maladevi.

In addition to these serious slips slight trouble has occurred at the toes of Shirsuphal Bhadalwadi and Matoba, and a most instructive slip occurred at Pashan.

Probably there have been slips at the toes of other tanks, but I have personal knowledge, of all the above and have visited all the banks excepting Nehr.

Now, there must be some reason for these persistent indications that the banks are unsafe; it may be said they are due to bad material or bad workmanship, but they are so universal and the banks have been made by so many different men with all the care they could give, that the slips can hardly be due to any local defects of construction, but must be due to some general case showing its results first at the weakest part of the banks.

Now, if the cross sections of all sorts of embankments be studied, from that of a small water-course up to a small tank embankment with 40 feet depth of water, it will be found that in all cases where the bank is thoroughly satisfactory the hydraulic gradient through the bank is not less than 4 to 1; the hydraulic gradient is the slope from the point where the water touches the bank to the rear toe, and is the maximum gradient available for driving the water through the bank.

Figures 3, 4, 5, 6 and 7 are types of banks with the hydraulic gradient 4 to 1, the depth of water varying from 2 feet to 40 feet; they are not extravagant sections, those for canals are less than the cross sections given in the plans for the Ojhar and Lakh canals now being commented on, and if works were constructed of weaker cross section then trouble would be expected at the rear toe of the banks.

If, however, we calculate the hydraulic gradients for the tank embankments which have slipped, we find that none of them are 4 to 1, but that all are steeper. The actual cross section and hydraulic gradient of Waghad, Mhasvad, Ashti and Pashan have been ascertained, and for the others calculated approximately from the comparative statement of storage tanks in

the Deccan, and the depths of water stored and hydraulic gradients are given in the table below :—

	Name of work.	Depth of water below weir crest.	Hydraulic gradient.
These banks slipped badly	Waghad	83	2·7 to 1
	Mhasvad	67	2·96 to 1
	Nehr	60	3·00 to 1
	Ekruk	58	3·48 to 1
	Ashti	50	3·09 to 1
These banks slipped at toe	Bhadalvadi	44	3·50 to 1
	Shirsuphal	48	2·70 to 1
	Pashan	40	3·4 to 1
		43	3·1 to 1
	Matoba	39	3·4 to 1

Pashan is most instructive, as it is a case where the depth stored was raised until the embankment slipped.

The slips at these banks seem to show that the limiting height for safety for the cross section adopted, had been decidedly exceeded for the banks with 50 feet depth or over, and just passed for depths of about 40 feet. When suitable berms are added at the rear of these 40 feet depth banks, the slipping is stopped. The addition of the berms reduces the hydraulic gradient to about 4 to 1.

Cf. American Engineering News, dated 28th November 1901, pages 410—413.

In addition to this Indian experience, valuable information is available in the report of the Board of Engineers in America, appointed to consider the proposal that part of the Croton Dam should be of earthwork. The Board took observations of the saturation of the earth of six earthen dams from 50 to 90 feet total height and found as follows :—

That the embankments were always completely saturated as far as the puddle wall; that moisture passed through the puddle wall with a loss of head of about 10 feet; that the line of saturation to the rear of the puddle wall varied with the material, when water-tight the slope was 35 per cent. or about 3 to 1, and where very porous 10 per cent. or 10 to 1.

They deduce from their observations that with carefully selected material a drop of 17 per cent. of the head in the reservoir may be expected at the puddle wall and a slope of saturation of 23 per cent. or 5 to 1 in the rear of the bank.

They finally reported as follows :—

"On this basis which is a liberal one the maximum height to which an earth embankment with its top 20 feet above water line, and with outside slopes of two to one can be built with safety is 70 feet." (Extract ends).

This cross section recommended is shown in figure 8, the hydraulic gradient is $\frac{220}{50} = 4{\cdot}4$ to 1. It is compared with Maladevi in figure 9.

This agrees precisely with our Indian experience noted above that the hydraulic gradient should not be less than 4 to 1 for banks up to 40 or to 50 feet depth of water.

For depths above 40 or 50 feet the pressure on saturated material becomes so great that it would flow unless held in place by dry material. Saturated earth weighs about 120 lbs. per cubic foot, and for a bank of the section proposed for Maladevi with a depth of water of 100 feet, the average pressure on the base due to the earth alone, if the weight were evenly distributed, amounts to 3 tons per square foot or 47 lbs. per square inch. For 50 feet depth of water it amounts to 1·70 tons or 26 lbs. per square inch.

The water pressure must increase this pressure on the base, the amount of increase is unknown, but it cannot exceed that due to the full head of water; and this would bring the intensities of pressure up to 5·8 tons per square foot or 90 lbs. per square inch for 100 feet depth and 3 tons per square foot or 48 lbs. per square inch for the 50 feet depth.

The true intensity is somewhere between these two extremes and probably nearer the lower limit, but the lowest pressure of all for the 50 feet depth is 26 lbs. per square inch and is sufficient to make saturated clay flow unless held down by dry material, but in the cross section of the Deccan tanks there is little or no dry material at the toe when 40 feet depth is exceeded.

This seems to me to sufficiently explain the movement which has occurred at the rear toes of nearly all Deccan tanks with over 40 feet of water and to explain the slips which have occurred in the higher banks.

In the Maladevi cross section instead of the customary plan of providing a sufficiency of dry material to weight the saturated stuff and keep it from moving, it is proposed to receive the thrust on a dry stone toe with a top slope of $2\frac{1}{2}$ to 1, the same as the rear slope of the bank.

The intensities of pressure on the saturated earth are at 50 feet depth somewhere between 26 and 48 lbs. and at 100 feet depth between 47 and 90 lbs. per square inch, and when the flood waters back up the river and penetrate through the dry stone the saturation will be increased, and it seems probable that the pressure will then be nearer the higher than the lower limit.

It seems to me that the soft clay and saturated earth must be driven into the spaces between the stones, and in time will completely fill them, so that the stones will no longer act as a drain, but will be a mass of stones in slipping mud, much like the general body of the dam and liable to slide along with the whole mass.

I do not think the dry stone toe will be sufficient to support a dam of so weak a section.

The cross sections of the American dams which were examined by the Board on the Croton Dam referred to above, are stronger than the Deccan section, and I have given the names and approximate hydraulic gradients in the table below:—

Table of Hydraulic gradients of American High Earthen Dams.

Reference.		Names.	Depth of water stored.	Hydraulic gradient.
Scaled from drawings in *American Engineering News* of 28th November 1901		Titicus (New York)	From 40 to 70 feet depth of water.	4 to 1
		Bog Brook		4 to 1
		Carmel Main Reservoir D.		4 to 1
		Middle Branch		4 to 1
		Amawalk Dam		5 to 1
Ditto	11th September 1902 page 187.	San Leandro San Francisco	117 reduced by silting to 85.	7 to 1
Ditto	20th February 1902, page 159.	Druid Lake, Baltimore	82	3 to 1
Ditto	10th July 1902, page 26.	Tabeaud Jackson, California	92	3·2 to 1
Ditto	9th October 1902, page 290.	Utica, New York	62	2·2 to 1 and breached.

The Druid Lake and Tabeaud dams are in very narrow gorges and the total length of the dam is equal or less than the width of the base so that the sides of the gorge must support to some extent the centre of the dam. Anxiety was expressed about them and a Board appointed to examine them reported them safe.

San Leandro near San Francisco is frequently quoted as a very high dam. It is 125 feet total height and water is within 8 feet of the top, but the great rear slope is not always mentioned, a portion of it equal to the Maladevi section was made of selected material carefully constructed and then the rear filled in to 6·7 to 1 by hydraulic sluicing. It is a narrow gorge only 500 feet wide.

Cf. American Engineering News, 9th October 1902, page 290.

A most instructive report is given of the failure of the earth dam at Utica, New York. The dam was 70 feet high, rear slope $1\frac{1}{2}$ to 1, inner slope 2 to 1, top width 15 feet.

The hydraulic gradient was 2·2 to 1 only. Slips occurred from the first, and finally the dam failed, and after water had quickly run through the breach the inner slope of 2 to 1 collapsed.

Now, if the failure be compared with the slips in the Indian Deccan banks, it is seen that for an hydraulic gradient of 2·2 to 1 the bank failed and breached. That for hydraulic gradients of 2·5 to 1 to 2·9 to 1 the banks slipped badly. That for hydraulic gradients of about 3 to 1 the toes slipped slightly with 40 feet depth. These facts, learned by experience, seem to show that the cross section of the earthen dams in the Deccan is at the very minimum limit of safety with 40 feet depth of water.

Now in iron work or masonry work a factor of safety is always used, generally about 5 or 6 to 1, and it seems to be but reasonable to adopt a larger factor of safety for great heights of dam than for smaller ones, for the stresses in the earth are more uncertain and the costs of failure greater. For dams of height greater than 40 feet it will be but ordinary precaution to have 20 feet of dry material above the average limit of saturation, and the American Board's line seems a fair average. This in practice will make the rear slope of a high dam 5 to 1 if the top width be 10 feet and top 10 feet only above water level; see figure 10.

For dams with 40 feet depth only the precaution may be less and a slope of 4 to 1 will be sufficient; see figure 11.

All the above notes refer to dams of the average materials available for earthwork, namely, earth and moorum.

PAPER No. 18.

Proposed high masonry dam at Bhandardarra.

The Bhandardarra Dam is proposed to provide storage for the Ojhar Right and Left Bank canals in the Pravara Valley, of the Ahmednagar District, in place of the Mahaladevi Earthen Dam the foundations of which have been proved to be bad. The site is at a narrow gorge near the head of the valley, the fall of the river bed is steep and up to 150 feet height the storage obtained is small and expensive; but above this height the rate per million cubic feet becomes cheaper as the dam is raised until at 250 feet height, 8,671 millions are stored at a cost of Rs. 30,23,974 per million for works only.

The storage and cost for various heights is as follows:—

Height.	Storage in million per cubic feet.	Cost in Rupees.	Rate per million cubic feet of waters stored.
200	3,331	16,20,158	486
210	4,100	18,60,000	454
220	5,000	21,15,000	423
230	6,017	23,92,085	388
250	8,671	30,23,974	349

The width of base and cost of the dam depend upon the maximum intensity of pressure allowed on mortar made from the kankar lime available in the District. The kankar for the large Masonry Dam for the Tansa Water Works was obtained from the District around Bhandardarra, and a very large number of experiments on the crushing strength of mortar were made under the M. Clerk's direction on the Tansa works.

A table of results is given in volume CXV of the Minutes of Proceedings of the Institute of Civil Engineers, pages 40 and 41. Tests were made constantly on the work, and the average samples were sent to England every year and tested by the specialists Messrs. Kirkaldy & Sons. Mortar was made by steam mills and also by the ordinary bullock ghani or edge mill.

The steam mills gave the better results. For the lower part of the dam special care was taken to provide clean sand and for this mortar a crushing stress of over 2,000 lbs. per square inch was obtained. The average results for mortar more than two years old was over 1,500lbs. The mortar was made of 2 lime to 3 sand by measure. At Bhatgarh Dam, Lake Whiting, the proportions of the mortar were 1 of lime to 2 of sand and a great many tests were made, the average results were 821 lbs. per square inch, many results being over 1,000 lbs.; steam mills were not used at first at Bhatgarh, and these results are mainly for mortar made by bullock mills and the briquettes were 2 inch cube and crushed under a lever made from a piece of rail.

The maximum intensity of pressure on the dam was 150 lbs. per square inch. The factor of safety is thus about $5\frac{1}{2}$. At the Croton Dam, the pressure is, I believe, about 230 lbs. per square inch.

If mortar be made in the same proportion as at Tansa, 2 lime to 3 sand a factor of safety of 5 be taken, and then for mortar made with special care in steam mills a pressure of 400 lbs. per square inch would be safe; or taking the average Tansa strength, 300 lbs. would be safe. In the Bhandardarra Dam the maximum pressure has been kept below 210 lbs. per square inch. So that for special steam mill mortar the factor of safety would be over 9 and for the average results over 7.

Further this maximum pressure does not occur all over the base of the dam, but is a theoretical calculation of the extreme maximum at the toe and the elasticity of the material will distribute and reduce the pressure.

Two hundred and ten lbs appears quite safe and 250 lbs. might be used if necessary without danger in designing a higher dam. For this special mortar the sand must be washed quite clean and test adopted by the author is as follows :—

A pint of sand to be placed in a basin 18 inches diameter and 6 inches deep filled with clean water. The sand to be stirred up thoroughly. It should then be possible to see the sand through the water.

If the water be discoloured so much that the sand is not visible the sand is sent back to be washed again.

The Deccan sands are coarse and with running water can be easily washed to the above requirements.

For Bhandardarra provision is made for constructing a small temporary dam to give a supply of water until the main dam is raised high enough to store sufficient water for the work.

Steam mills must be used as the space is limited and the dam is a large mass of work concentrated in a short length of 800 feet.

As rock is always plentiful at the site of a masonry dam it is cheapest to make the base of the dam of rubble masonry laid in as large blocks as can be conveniently handled well bedded in mortar, the spaces between the stones to be carefully filled by first putting in mortar and then pushing chips or small stones into the mortar. It is very necessary to insist on this for if stones or chips be laid first and then an attempt be made to pour mortar between the joints hollows will result; the work must be built of mortar with stones bedded into it.

In quarrying the rubble for the lower part of the dam a quantity of small chips will result and these may be conveniently broken up for concrete and the upper part of the dam made of concrete.

The face work of the dam should be of uncoursed rubble, for if coursed work be adopted it is necessary to obtain special stones for the face work, and to provide special skilled labourers for laying the work, and for cutting the stones; the courses also require to be set out and levelled. Obtaining these stones and laying the coursed work, delays the construction of the dam most seriously, the author was on the construction of the Bhatgarh and Tansa Dams and in both the rate of progress would have been much more rapid, if it had not been necessary to wait for face stones.

At the lower part of the dam, where the batter of the face is great, it is necessary to build the uncoursed rubble in layers at right angles to the face for a few feet, joined by a curve to the mass of the dam, unless this is done, very long stones would be necessary at the face to give bond to the work and there would be much waste in cutting the face of the outer stones to the batter.

Outlets.

The outlet sluices for great depths are a difficulty, the pressure for 150 feet depth is nearly 4½ tons per square foot.

Sluices are provided at various depths and when issuing water it would not be necessary to work against great heads, but it is quite possible that when the tank is low at the beginning of the rains a sudden and heavy flood may raise the water level quickly, and it is, therefore, advisable, if not necessary, to arrange to close the sluice gates under the maximum head. It is, therefore, proposed for depths greater than 76 feet to lay pipes 36 inches diameter through the dam and place sluice valves at the downstream end where they are readily accessible. In addition to these downstream sluice valves, it is proposed to provide at the upstream end of the pipe slide valves or gates to be worked from the top of the dam and to fit them with a small reducing valve; a one-inch cock would be sufficient. The downstream sluice valves are to be fitted with gear to close them against the maximum head

of water. The up-stream slide valves would be worked when the downstream valves are closed, and so, would be worked in still water and not under pressure the ordinary long rod with a screwed length and nut at the top of the dam where it can be kept clean seems the simplest arrangement for these upstream slide valves.

Waste weir in Gorge.

The waste weir is another interesting item of the proposed works. The site had been previously rejected because there were no suitable waste weirs and at that time the Tansa and Bhatgarh tests on the crushing stength of mortar had not been made and the Engineers there were unwilling to recommend dams of over 120 feet in height.

With a dam 250 or 260 feet high a waste weir can be made on the ridge on the south bank in the ordinary way but for lower heights of dam, where it would be costly to cut down the ridge, a good waste weir can be obtained at any height by throwing back the end of the dam upstream nearly along a contour line, any desired length of weir can thus be obtained without a contracted approach channel, but with free flow of water from the depth of the tank.

The water will run down the face of the hill side with great velocity and in calculating the size of the discharging channel required a high velocity may be taken. The water after falling over the weir has a considerable velocity, but neglecting this and assuming that it has to be re-started, a head of 5 feet and co-efficient of 0·62 would give a velocity of 12 feet per second. If the weir be made 20 feet high, then deducting the 5 feet, there will be 15 feet depth available for discharge and the water way to be excavated in the hill side will vary from a little over zero at the far end to $\frac{\text{Discharge}}{12 \times 15}$ at the exit end of the channel, the channel should then be continued with fall to maintain the velocity and a side wall to keep in the water until it can fall clear of the toe of the dam.

The plan and section of such a waste weir are shown in Plate 35 and require much less excavation than when both approach and discharge channel have to be excavated for a broad width of weir. Further, the material to be excavated in the Bhandardarra case is all good rock suitable for use in building the dam ; the excavation therefore would not cost anything but would be a convenient quarry for the upper part of the works.

The author believes that this type of weir can frequently be adopted with economy both for masonry and earthen dams.

PAPER No. 19.

On Narora Weir.

The Lower Ganges Canal takes off at Narora. The construction of the works commenced in 1873, and the canal was opened for irrigation early in 1878.

At the weir crossing the bed of the Ganges consists of very fine sand, under which a bed of sound clay lies at an average depth of 29 feet below the weir floor.

The design for the weir caused some controversy: the one party demanding deep foundations, and, if possible, a water-tight section to be obtained by carrying the lower curtain wells down into the clay bed and closing the apertures in both lines of wells by wedges of clay puddle on the upstream face, in order to meet any retrogression of bed levels and to prevent leakage below the floor. The other side favoured shallower foundations as sufficient, holding that in such a river retrogression would be confined mainly to the stream below the weir sluices, and could be met as it occurred by an extension of the talus; and that with the construction of the weir, leakage would be automatically lessened and finally closed by the deposit of clay in the bed above.

The design adopted (see cross section on Plate 36, as actually constructed was a compromise; the proposed depth of wells was reduced, but the deeper wells are placed in the lower curtain; the puddle was omitted as unnecessary.

The crest wall 10 feet high, with drop gates of 3 feet height on top, is founded on well blocks 10 feet square sunk to R. L. 565, or 7 feet below the floor; the intervals between these blocks are closed with sheet piling and filled with concrete, forming a fairly solid wall. The floor 40 feet wide is 5 feet thick, including the ashlar covering. The curtain wells at the toe of the floor are of 8 feet diameter sunk to R. L. 555, or 17 feet below floor, except in the first 500 feet adjoining the sluices, where they are carried down into the clay; the intervals between these wells are but indifferently closed with piles.

A cistern wall 3 feet high was provided at the end of the floor, but omitted during construction.

During construction in 1876 the river floods were passed over a portion of the work, the wells and flooring having been completed and the crest wall raised one foot above the floor. After the floods it was found that the floor had cracked, and the crest wall subsided slightly in a length of 100 feet. This was caused by the draw along the weir face tow ards the sluices, the evidence from the displaced pitching showing that the scour had gone below the block foundation.

It had not been intended to protect the upstream face with anything but a short apron of plain pitching laid on a counter slope to the crest wall. This accident, and the failure in the same year of another work of similar design showed the necessity of further protection upstream. The base of puddle was then introduced and the apron laid as shown in the plan.

The downstream talus was at the same time extended to 100 feet width the longitudinal and cross walls of concrete built to localize scour, and the top layer up to the concrete wall laid with specially large blocks and grouted with concrete. The extra length of 30 feet was laid three years after the opening of the canal as a further protection.

With the opening of the canal the draw towards the sluices becomes more pronounced. It was combated by the erection of a groyne on the left flank of the sluices and a number of spurs at intervals along the weir face. These spurs of block kankar were generally of 100 feet length, but too weak in section and were speedily demolished. An island that formed upstream of the weir was also protected and utilized to break up the flood stream and to prevent scour. Such measures were of very temporary utility; scour holes were

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on several occasions found in perilous proximity to the weir crest, until in 1896 two permanent groynes were built, the one on the left flank of the weir sluices, the other at about the centre of the weir. These groynes of 568 feet length and 48 feet top width, are of sand strongly rivetted with block kankar on the slopes.

Intermediate groynes of the same pattern were constructed after the failure of the weir floor. The groynes have proved very stable and of great utility in controlling the floods and keeping scour at a distance from the weir face.

Springs appeared in the weir talus immediately after the opening of the canal, generally through interstices in the grouted portion; there is no record of any serious outblow of sand, no danger was anticipated from them, nor, so far as is known, had any been observed in the length that subsequently failed. In the course of years a retrogession of levels has occurred, and the low water surface was 3 feet lower than at the time of construction.

In March 1898 Mr. Beresford, Inspector General of Irrigation, when visiting Narora, noticed some springs throwing up sand. This fact and the results of some experiments carried out at the time showing an excessive pressure on the floor, pointed to the necessity of strengthening the work and reducing the pressure by the addition of a puddle apron upstream, a necessity clearly proven by the failure, within three days of his visit, of a portion of the floor.

At the time of the accident, 30th March, the water surface above the weir was at R. L. 584·30 and downstream at floor level, giving a head of 12·2 feet on the floor; at the point of failure the grouted talus was particularly good and extra pressure was thereby thrown on the flooring. A strong spring burst through the floor at the toe of the crest wall (the joint between the well blocks and the concrete floor), and, passing under the stone flooring, lifted it bodily over a length of 340 feet to a maximum height of 2·23 feet. The top layer of the brick work bedding was also blown up at the point of greatest disturbance. The weir wall settled in a length of 120 feet to a maximum of ·29 foot, the flooring showing vertical cracks. Downstream the grouted pitching was blown up, and a hole, 8 feet deep, formed, and elsewhere in the disturbed area it was shaken.

Upstream the whole of the apron had disappeared, and the wall was exposed to a depth of 7·5 to 9 feet, and showed a horizontal crack, running over 3 chains length and 6 feet below the crest

Borings through the floor revealed cavities below the concrete, extending to about 50 feet on each side of the point of fracture, and as much as 8 feet in depth at the point where the spring burst through the lower line of wells and out into the grouted pitching. The whole length of the weir floor was subsequently tested by boring, but no other hollows were found.

The works undertaken for the protection and strengthening of the weir were—

(*a*) An upstream apron of puddle covered with pitching, 100 feet wide.

(*b*) A line of sheet piling along whole length of weir.

(*c*) Two intermediate groynes on upstream face.

(*d*) A dwarf wall 3 feet high on the floor of the weir.

In the short period available for work between the date of failure and the setting in of the rains, it was only possible to complete a length of 900 feet of the apron covering the injured part of the weir. For the same reason the repairs were restricted to a length of 100 feet where the floor had been most damaged. The whole of the injured length was protected from direct flood action by a stout bund along the weir crest, raised three feet above high flood level, and by cross bunds on the downstream side.

In the second season's work, the apron and groynes were completed, three-quarters of the length of dwarf walling, and one-quarter of the sheet piling.

In the repairs to the damaged floor, the hollows below were filled with semi-liquid clay, poured in through borings, the springs and cracks were staunched and closed with fine cement concrete, the thickness of the floor was increased by two feet of brickwork at the centre and one foot at the ends.

In laying the dwarf wall on the floor a good deal of trouble was met with from numerous small springs on the left half of the weir; there were also several cracks through which water passed freely. No hollows below the floor could be found, the springs and cracks were closed, and by the construction of the dwarf wall forced outwards. The sheet piling on the left half of the weir was also driven $1\frac{1}{2}$ feet deeper.

The upstream apron consists of $2\frac{1}{2}$ feet puddle of good brick clay, thoroughly worked up at site in 6-inch layers. In the second season's work a layer of river sand, 1 foot thick, was laid over the puddle, to prevent any subsidence of the overlying pitching, some signs of sinkage having been noticed in the portion first completed.

The pitching over the puddle is 2 feet thick, and divided into compartments by walls of block kankar laid in lime; the top layer of the pitching is of specially large blocks securely packed.

The upstream face of the whole apron is protected by a wall of kankar masonry, 5 feet thick, carried 6 inches below the bed of the puddle, and by a strong berm of kankar pitching, 15 feet wide on the surface.

The sheet piling is of sâl timber, $4\frac{1}{2}''$ thickness, tongued and grooved with guide piles ($12'' \times 9''$) at 12 to 14 feet intervals, at a distance of 70 feet ($78\frac{1}{2}$ opposite the injured portion) from the crest walls. On the left flank it is connected with the weir revetment, and in that half of the weir is driven to a mean depth (R. L. 558·50) of 13·5 feet below weir floor. On the right half it is driven to R. L. 560 mean and is connected with the left revetment of the weir sluices. The pile engines used were of the rail pattern as adopted at Khanki weir, and with them the daily progress of work was from 40 to 50 running feet.

In addition to the works already noted, a slab of concrete was laid below the dwarf wall on the floor, from the whole length of the weir; the conditions of the river did not permit of its being carried out before February 1902. The slab of concrete is 25 feet wide and $1\frac{1}{2}$ feet thick, with a cistern of 15″ depth on the top, divided off by brick walls into compartments of 200 feet length. The cleaning of the floor and talus, preparatory to laying the concrete, brought to light a large number of springs, which had been hidden by the bajri and sand deposit; these were generally found at the junction of the stone flooring and the talus. On the right half of the floor no great difficulty was experienced in closing the springs, but on the left half, owing to the heavy silt deposit on the talus below the floor, the pressure was much greater, and springs rose to 3 feet above the floor before they could be killed. The completion of the concrete forced the springs outwards to new vents in the talus below; in no case was there any sign of sand outblow through them.

The works generally have stood very well; there have up to the present been no signs of any subsidence in the upstream apron. The river action is occasionally very severe at the heads of the groynes, but all scour is well away from the apron, and, beyond renewals to revetting of the noses of the groynes, no further work has been necessary. The flooring is in good order, though there are some slight signs of increase of springs.

In 1898 Mr. Beresford instituted some tests of the pressure on the weir floor. The observations of these have been continued during periods of low river.

At three points in the length of the weir, pipes have been inserted as shown in the plate.

For the purpsse of testing the efficiency of the apron and sheet piling it is unfortunate that more extended observations of the pressure had not been made, as we have but one set to show the pre-failure conditions. The head on the floor has also been limited since the failure of the work, and more recently has been reduced by the maintenance of a backwater on the talus. The

observations tabulated below have been selected, to give the results for approximately equal heads during the years since 1898, and show that, though there was no remarkable drop in the pressure on the completion of the apron, there has been a steady and progressive improvement in the reduction of pressure on the work.

As work was in progress during the cold weather of 1898-99 and 1900, most of the observations taken during those periods are of no critical value.

The decrease in total head noticeable from the right to the left flank of the weir is due to the heavy silt deposits on the left flank, which hold up water on the floor.

NARORA WEIR.

Pressure tests.

Date of observation.	R. L. W. S. above weir.	Head.	Average R. L. water in pipes.	Loss of Head.	Proportionate loss.	REMARKS.
LINE NO. 1.—CHAIN 33.—RIGHT FLANK OF WEIR.						
20th March 1898	584·2	12·2	77·6	6·6	5 06	Before failure
11th October 1899	84·01	9·13	79·07	4·91	5·05	Apron laid.
9th May 1900	84·35	9·35	78·70	5·65	5·68	Piling completed.
11th December 1901	81·88	10·8	75·33	6·55	5·7	...
22nd December 1902	82·20	10·0	75·75	6 45	6·06	Concrete slab laid below dwarf wall on floor.
3rd February 1903	81·70	9·25	75·52	6·18	6 28	
31st May 1904	83·50	9·33	77·12	6·38	6·43	
LINE NO. 2.—AT CENTRE OF WEIR.						
8th May 1900	83 20	8·8	78·04	5·16	...	
11th December 1901	81 88	10·86	75·33	6·55	5·38	
22nd December 1902	82 20	9·05	76·70	5·5	5·36	
3rd February 1903	81·70	8·5	76·45	5·25	5·44	
31st May 1904	83·60	8·68	77·55	6·05	6·13	
LINE NO. 3.—AT CHAIN 9.—LEFT FLANK OF WEIR.						
11th April 1900	80·90	7·55	76·28	4·62	4 87	
28th April 1901	80·92	7·77	75·25	5 67	5·85	
22nd December 1902	82·20	8·43	76·37	5·83	5·06	
10th January 1903	81·70	7·97	76·28	5 42	5·41	
31st May 1904	83·70	8·07	77·14	6·56	6·50	

It is certain that for many years the weir was subjected to undue strain from currents parallel to the face. The upstream apron and the spurs were far too weak to check this action.

In 1878, the first year of trial, it was found that the apron had been much displaced, and in places had sunk to 3 feet below the original bed of the puddle. As late as 1895 a depth of 35 feet was measured, during the floods, within forty feet of the weir wall. Such scour must have disturbed a good deal of the weir and well foundations, and it is only a matter of wonder that a more general and more serious subsidence had not occurred.

The dominant idea in the designs of a large number of works executed about this same period was the necessity of protection against scour on the downstream side; hence the deeper wells on the lower side and the heavy talus. Protection from percolation under the works was anticipated by the construction of perfectly water-tight joints between the wells, and this was never attained.

At Narora there is but little scour on the downstream side, and there never has been any serious displacement of the talus; such scour as does occur is below the sluices, and this can be checked by judicious manipulation of the sluices.

The moral of the history is that for the protection of a work like the Narora weir an ample upstream apron is essential, and that currents parallel to the work must be prevented, and scour kept at a distance by groynes on the upstream face.

PAPER No. 20.

On failure of a 14 feet Fall on Nadrai Escape, Lower Ganges Canal.

The Nadrai Escape constructed in 1878 was designed to carry 2,400 cusecs, the descent from the canal to the river being disposed of by a fall of 7 feet through the head work and a fall of 14 feet, the subject of this note.

The work was always considered somewhat of an experiment and was from the first a source of anxiety. Built in light sandy soil, subjected to heavy spring action and to a gradual retrogression of levels in the stream below, it failed to meet the test.

For two years previous to the failure, the work had been severely tried by the necessity of working the Escape continuously for long periods. The Nadrai Aqueduct over the Kali Nadi was under construction and it was necessary to maintain a supply in the upper reach of the Canal for boating and for the irrigation on the Farrakhabad Branch. During these periods as much as 2,000 cusecs was on occasion passed down the Escape, the maximum head on the fall at any time being 20·3 feet.

With a length of 120 feet the crest wall was carried on well blocks sunk 7 feet below the floor level: the downstream curtain wells being carried to 12 feet depth. The floor consisted of 3 feet concrete covered with stone slabs 1 foot thick and was only 22½ feet wide. Upstream a puddle apron 43 feet wide and 1′—9″ thick was provided but protected with pitching for 25 feet only from the crest of the fall. The downstream talus was of kunkar pitching.

The first signs of failure occurred in 1887, when some of the floor stones on the lower edge were displaced and the floor cracked all along at the junction of the concrete and downstream wells. The pitching below the floor was scoured out to a depth of from 1½ to 2½ feet. Two strong springs burst out a short distance below and opposite spaces between the lower wells. An examination of the floor showed that there were cavities below up to 1½ feet depth.

To strengthen the work an additional 15 feet of flooring 5 feet thick (concrete 4 feet brickwork 1 foot) was added and the wings were extended 30 feet downstream. The cavities below the floor were stock rammed with pure kunkar lime and to all appearance completely filled. The talus was re-laid, the top layer grouted in a width of 30 feet, and was extended to a distance of 80 feet below the floor.

Springs still gave trouble and some of the brick work on the new floor was blown up. A cistern wall 2 feet high was then added at the junction of the old and new work.

In October 1888 after the Escape had been running for a month continuously, the new cistern wall was blown up and carried away; some of the floor stones were washed out and others upheaved. The left wing wall cracked from top to bottom.

On first inspection this appeared to be the total damage, but twelve days later there came a sudden collapse. The water held up by the crest wall burst through, passed under the wells and floor, carrying with it lumps of clay and kunkar, blew up the grouted talus below, and wrecked the adjoining piece of the new floor.

The lower part crest wall in a length of 30 feet settled 8 inch, breaking away from the upper part which except for cracks and a slight sinkage, held firm. The left wing cracked in various places and gradually moved forward.

Towards the right flank and below the floor a well curb was found; the only well from which it could have come were one on the upstream wing above the crest and the other in the old flank wall. It must therefore have been carried out under the curtain wells, a testimony to the strength of the under-blow.

The wreck was complete and the only remedy was to construct a new Escape with a series of falls with smaller drops.

The quality of the original work was excellent, the concrete was sound, and all material of the best. Close to the work there appears to have been ample puddle though the apron did not extend sufficiently far upstream. The downstream pitching as re-laid in 1887 was extensive and good. The floor and wings were too short and the thickness of floor inadequate nor was any provision made to meet the retrogression of levels in the river below. There must have been considerable piping between the wells and under the floor which undetected and continuous brought about the catastrophe.

In a number of works of similar design with upstream and downstream wells, the joints between the floors and the wells have proved a weak spot, opening with the smallest settlement of the work and admitting excessive pressure to the floor covering.

Deep falls have been built in many places on the canal system and have in a great many cases given trouble from the percolation through and round the works.

In the comparatively light soils generally met with at their sites, they are dangerous and in no case are they economical. The construction of such falls have been abandoned for many years and a series of falls with small drops have been adopted.

On the new Nadrai Escape three falls were built, the two upper ones of 7 feet drop and the lowest one of 9 feet. The designs of all are similar except that in the lowest fall the cistern wall has been omitted and the concrete base increased to 3 feet thickness. The floor of the lowest fall is low enough to provide for any future retrogression of levels below.

The upstream puddle apron in each case is of 100 feet length and is protected by kunkar or brick pitching. The downstream talus though short is fully protected from undue scour by the back water from the fall and river below. These falls have up to date proved sound and efficient.

PAPER No. 21.

Self Acting Module for Regulating Irrigation.

The discharge through a canal outlet varies with the depth or level of water in the canal with reference to the outlet. The depth depends on the volume admitted into the canal, which varies with the area, distance and description of crops under irrigation in each season, and the system of water distribution in force.

To ensure a uniform rate of delivery through a given outlet, water should issue under a constant head of pressure. The water module here described is designed for delivery of water at a uniform rate, notwithstanding variations of level in the canal or basin of supply.

Description.

The module shown in the drawing consists of a hollow watertight iron tank floating in the water, and provided with submerged orifices on the sides into which water flows under a constant head. The water which enters the orifices is led into a flexible tube, attached to the bottom of the tank, whence it issues into an open channel.

The float, and the flexible tube fixed below it, move up and down in a perfectly vertical direction, the motion being so restricted by means of suitable guide bars. The friction of motion between the float and the bars is reduced to a minimum by guide wheels or roller bearings.

The flexible tube is the principal novelty in this appliance. In the particular form illustrated on accompanying drawing (Plate No. 38), the tube is a hose made of canvas, leather, or other similar suitable material, which is kept distended by means of a number of wooden wheels (similar to the wheels of a cart or a bicycle) with their naves moving up and down on a concentric vertical axis. The bearings are bushed with brass. The pressures round the hose which are horizontal are balanced; any unbalanced inequalities, being taken up by the central axis, have practically no effect on the action of the float.

The naves have an appreciable length of bearing to prevent the wheels from deviating from a horizontal position.

In the section (Fig. 3), the level of water against the float, measured above the bottom of the float, is practically always constant. The flow into the orifices *a*, *a*, being under a constant head, is always uniform. Water passes from the bottom *b* of the float with a free overfall into the flexible tube whence it issues into the outlet channel, *c*, *d*. Pieces of hose are attached to the lower ends of outlet pipes at *b* to break the force of the falling water. The tube has flexible folding sides. With every rise or fall of water level in the chamber, the tube elongates or contracts as in a bellows or a concertina.

In Fig. 1 is exhibited an arrangement for attaching a module to an existing canal outlet. Under this design water may be delivered at pleasure for irrigation either through the module, or independently of it when the module is not working.

The entrance to the module chamber consists of one or more apertures with an iron grating in front and grooves for stop-planks and puddle for cutting off the supply when necessary. A transverse partition screen is placed opposite the entrance to deaden the velocity of water.

Pipes will be inserted to provide an outlet for the air entrained in the flexible tube by the falling sheet of water.

The module is enclosed in a locked chamber so that no one may tamper with it while in use. In the particular design sketched in Fig. 2, it is assumed that the discharge from the main sluice is larger than the module can deal with, and there is no loss of head between the water in the canal and in the chamber.

The pressure of water against the bellows tube or hose will ordinarily be that due to the difference of water level between the basin of supply and outlet channel. Under ordinary conditions, the pressure will probably vary from 3 to 6 feet.

Folding tube or hose.

3 to 10 feet pressure against a canvas hose is inappreciable when it is remembered that a hose for fire service is often subjected to a pressure of over 500 feet.

A hose is perishable. But spare hose of standard sizes may be kept in stock at all times and slipped over, and lashed to, the rims of horizontal wheels or rings whenever an old hose is found leaky. This is the only part of the mechanism easily perishable, but it is also the part least expensive to renew as often as may be necessary.

The module as shown in the drawing is made of three principal part's namely, (1) an iron float with guide bars, (2) sluice valves and (3) a tube with flexible sides. These three parts may be made of standard patterns for any given size of module so that they may be interchangeable.

Standard parts.

The modules may be made of fixed sizes, determined after sufficient trial, to save cost and delays in manufacture. The size may be fixed for mean discharges of, say, 2, 5, 10, 15 and 25 cubic feet per second respectively.

By weighting or relieving the float, the discharges are capable of being increased or reduced within wide limits. Reduction may also be effected to any desired extent by throttling the orifices.

In a recent experiment, the float worked under a variation of head of 3 feet and the extreme variation of discharge was 3 per cent. and the mean $1\frac{1}{2}$ per cent.

The variation can always be reduced to any desired extent by a proportionate enlargement of the float.

The design may be varied to suit individual cases. The module may be formed of two or more tube outlets, instead of one, attached to a large float. Or there may be two or more floats supporting one or more suspended orifices. The latter may be an open pipe attached to the upper end of the tube outlet and resting on girders or cross bars held up on either side by floats. The open pipe can be adjusted to any required level by screw gearing or otherwise.

Variations in design.

The module will enable the supply of water for irrigation or any other purpose to be charged for by volume. The quantity of water required for watering a crop varies with the length of the channel leading to it, the season, the nature of soil, etc., and the Indian cultivator is not sufficiently enlightened to be left to his own devices for maturing a crop. Wherever, therefore, cultivators are not prepared to pay by volume, there may be no departure from the immemorial custom to charge rates on the crop area irrigated.

Distribution to Consumers.

It is proposed to send round a trained gang of canal water-men periodically, say, once in two months, to test the time taken to water a given crop area under an outlet from which a constant discharge is maintained by means of a module. If the gang should take, say, 5 days to give one watering, the outlet may be kept open for $5\frac{1}{2}$ days to make allowance for imperfections in the distribution by the cultivators. The intermediate distribution may be made by themselves assisted by a water-man (*Patkari*) of their own selection and paid partly by the cultivators. The duty of water on each outlet can be definitely fixed from time to time in this way and the cultivators and *Patkari* held responsible for the intermediate waterings. This is the nearest approach possible at present to the assessment of water by a volume.

The module can be tested by closing the inlet (Fig. 1) to the module chamber and noting the time taken to empty the chamber by a given depth. This gives the correct rate of discharge through the module. The test can be

Testing the module.

carried out to the satisfaction of all intelligent cultivators or their employers. Each module has in this way an almost perfect means of adjustment, re-adjustment and check.

The level at which water can be delivered into a distributary is of prime importance. The loss of head or difference of level between the full supply in the canal and in the distributary should be as small as possible.

Loss of head.

The maximum loss of head may usually be taken equal to the depth of water against the float *plus* 1 foot. This may be reduced by increasing the horizontal dimensions of the float and the diameter of the flexible tube. Up to a discharge of 10 cusecs, in ordinary practice, the loss of head measured below the canal surface level need not exceed 4 feet.

The minimum loss of head in the experiment referred to above was 2·30 feet.

A rough design of this module was submitted for the sanction of Government *before* the issue of the Report of the Indian Irrigation Commission. It was designed for the distribution of the water under the block system described in paragraph 291 of Part I. of that Report.

The Commission has remarked (*vide* paragraph 288) that *a system of long leases based on charges by volume will be very suitable for some of the works in the Bombay Deccan. It is probable that distribution by modules would result in great economy, even if the people preferred to adhere to the present system of assessment. The more systematic the distribution, and the greater the certainty of the cultivator as to the supply he will receive, the greater will be the efficiency of the canal, whatever system of assessment be adopted.*

The Commission was of opinion that great improvement in the duty and expansion of revenue *will be obtained when the distribution is regulated by more accurate measurements, such as are now recognised as essential to real progress in every Department of practical science, and in a great many industrial undertakings.*

The Commission recommended that strenuous and continuous efforts should be made to perfect the system of distribution, by the use of modules and other means.

On the Deccan canals where the storage is very costly, economy in the use of water is of the highest importance. Under more scientific regulation there will be less waste, and the same supply will irrigate a larger area and bring in more revenue. The cultivators will share the responsibility for distribution of the water after it is parcelled out at the outlets. The module provides them with the means of satisfying themselves, if they choose, that they receive the supply they are entitled to. To the canal officer the module provides a standard measure for distributing and regulating the water with an exactitude not hitherto attempted in the Deccan.

PAPER No. 22.

Lift Irrigation.

With a few trifling exceptions no irrigation has been carried out in India by means of pumps. This is the more surprising, since in Egypt where irrigation is being developed by Indian Engineers, 4,000 steam pumps are annually employed in lifting water for half a million acres. Moreover, in India, more than one-fourth of the entire irrigated area is supplied by lift from wells by such primitive means as the mhôt, the picottah and the Persian wheel.

It is commonly accepted that an acre of food grain will support $2\frac{1}{2}$ persons for one year, and this figure has been adopted by the Irrigation Commission when they say that ·4 of an acre per head of population must be protected in any given district if it is to be considered free from famine.

Taking the figures for British India, for which alone reliable statistics are available, the area annually sown is 226 million acres, of which 44 millions are protected by irrigation, while to come up to the above standard the irrigated area must be raised to 88 millions. If, therefore, it were possible to protect India from famine by irrigation alone, it would be necessary, on the assumption that all irrigation is fully protected in all years and that it is evenly distributed over the country, to just double the existing area.

In the Punjab, Sind and Burma considerable extension of irrigation by direct flow is likely to take place, while in Madras extensive schemes of storage are under investigation, but it is not likely that this increase will amount to more than from 8 to 10 million acres, and if the requisite 44 millions of increased irrigation are to be approached, extensive schemes of lift irrigation by the most approved mechanical methods must be worked out.

A scheme for the irrigation of 50,000 acres in Divi Island at the mouth of the Kistna has recently been submitted for sanction. Steam pumping plant was at first proposed, but further enquiry showed that a great economy in working expenses, amounting to 20 per cent., would be effected by adopting the Diesel Oil Engine. This engine burns crude liquid fuel and has no igniter of any kind. The air drawn in at the first stroke is compressed during the second to a pressure of 500 lbs. to the square inch, and resulting temperature of about 900° Fahr. which suffices to burn rather than to explode the charge of liquid fuel which is forced into the cylinder by an air-jet under still higher pressure. The result is a very even effort on the piston and a high economy, owing to the greater pressures employed and the cheapness of the liquid fuel. The unit adopted in this project consist of a twin-cylinder Diesel of 160 B. H. P. coupled direct to a 36-inch centrifugal pump. Six such units are employed, with a seventh in reserve, which suffice to lift 500 cubic feet per second to a mean height of ten feet. The project shows on paper a return of 10 per cent.

The working of one of these engines can be inspected near Nagpur, where it is used to drive a number of ginning machines, and has recently replaced a steam engine, which at 50 I. H. P. consumed 4·25 pies worth of coal per horse power per hour, while the Diesel at 90 I. H. P. consumes 2·38 pies worth of oil.

Openings for pumping stations on this large scale exist at other places in the Godavari and Kistna districts, and possibly also in Sind and on the banks of the Tapti, Nerbudda and Sabarmati. Pumping machinery can be erected and housed for Rs. 20 per acre. Channels and masonry works with the attendant establishment charges will come to Rs. 12, making a total cost of Rs. 32 per acre. Any such extensions, however, are not likely to exceed collectively one million acres, and for any material advance towards the additional 44 millions required, pumping schemes on different lines must be worked out.

Of the existing 44 millions of irrigated land nearly 30 per cent. are under wells, each irrigating on an average $4\frac{3}{4}$ acres. Taking a duty of 250 acres to the cubic foot, this means that each well discharges continuously less than $\frac{1}{50}$ cubic foot per second.

There are $2\frac{1}{2}$ million wells in British India and if the discharge of one-half of these could be increased five-fold so as to yield $\frac{1}{10}$ cubic foot per second instead of $\frac{1}{50}$, we should then have reached our 88 million acres of protected irrigation. When it is considered that probably one-half of these wells are in a porous soil and that two or even four teams of bullocks may be required in parts such as the United Provinces for a large well, it is not unreasonable to assume that in these situations the amount of water taken from a well is limited by the strength of the bulls rather than by the total quantity available. If the cultivator had more bulls he would probably dig more wells or get more water out of each, but bullocks are expensive to buy and to maintain.

The same result would be reached if he could be given some cheap mechanical power for raising his water.

Various estimates have been made of the cost of fitting a small pump and oil engine over a well where there was reason to suppose that an increased flow could be readily obtained, and assuming that a flow of say one cubic foot per second was available it is easy to show that such an arrangement would pay.

In Bikanir State a project is under consideration for lifting water for irrigation from wells 270 feet deep, by means of pumps electrically driven from a central power station. By reducing the unit of power to suit the requirements of an ordinary well, it is possible that a number of such schemes installed in suitable localities would go a long way in the direction required.

Mr. Chatterton has outlined a scheme of electric transmission of power from a central steam-driven plant; and, placing an electric motor and pump over each of a series of 40 wells, proposes to lift half a cubic foot per second for 14 or 16 hours a day from each well. As he says, however, these conditions of flow are favourable and unusual.

Taking $\frac{1}{10}$ cubic foot per second as a discharge likely to be realised from the majority of the existing wells and taking the average lift at 30 feet, then ·33 W. H. P. is the work done in lifting the water, which will require ·55 B. H. P. at the motor. An electric motor and centrifugal pump, directly coupled, could be fitted up over the well for say Rs. 500 and, estimating liberally for interest, depreciation and repairs at 15 per cent., the annual charge on this would be Rs. 75.

The cost of ·55 B. H. P. supplied continuously to the well may, as will be shown below, be put at Rs. 110 a year, making a total charge to the cultivator, who will own nothing but simply his well and the water flowing from it, of Rs. 185 per annum.

It may be assumed that the $\frac{1}{10}$ cubic foot per second continuous flow will be sufficient for 25 acres, and if it lasts for 9 months, say July to March, the owner of the well can grow two crops of cholum, cumbu, ragi, oil-seeds, vegetables, etc., and one crop of tobacco, saffron or sugar. The cost of irrigating one crop of wheat with water lifted by bullocks from a depth of 30 feet is estimated by Captain Clibborn at Rs. 11. It will, therefore, be a cheap bargain to the cultivator to charge him Rs. 8 per double crop in the case of cholum, etc., with a rate of say Rs. 20 for sugar and saffron, which are worth Rs. 200 per acre. The Revenue, therefore, at the lowest figure will be 25 acres at Rs. 8 or Rs. 200, or say Rs. 250 on the average. This allows of a large margin for expansion in the working expenses estimated at Rs. 185 per well. To do the same work by bullock power would cost several times Rs. 185.

In the Kistna Reservoir project now under investigation the total storage above the dam is 154,000 millions of cubic feet, of which two-thirds are required for the High Level Main Canal and the balance is to be passed down the river for use in the Kistna Delta. Combined with the normal discharge in the river this latter storage of 50,000 millions of cubic feet is sufficient to give a continuous flow, from the dam, of 3,000 cubic feet per second for 9 months, *i.e.*, from the beginning of July till the end of March. The maximum head available is 110 feet and the minimum is 70 feet, and 3,000 cubic feet of water falling 70 feet will yield 23,000 water H. P., available for Electric Transmission for any distance up to say 300 miles. Allowing for

all losses, it is usual to take a through efficiency of one-third, *i.e.*, every Water Horse Power at the dam will yield one-third of a W. H. P. at each well, which is the amount wanted to lift $\frac{1}{10}$ cubic foot per second a height of 30 feet. The 23,000 W. H. P. will, therefore, serve to supply 23,000 wells with one-third W. H. P. each; each well irrigating on the average 25 acres, or a gross area of 575,000 acres.

This half million, or more, of acres may be selected from anywhere within a radius of say 300 miles, but for economy of distribution they should preferably form one compact block. The driest and least protected, and consequently most famine-stricken region in the Madras Presidency consists of a block of some 8,000 miles in extent made up of 10 taluks of the Kurnool, Bellary and Anantapur districts. In none of these taluks is the ratio of protected irrigation per head of population greater than ·09, and in some it is much less, the standard ratio for protection being ·4. The figures for these taluks may be put into tabular form; and the area to be protected may be brought up to the desired total by taking on the way a second block formed of the two famine-stricken taluks of Cumbum and Markapur both in the Kurnool district :—

District.	Taluk.	Population.	Existing ratio of protection per head.	Area required to give a ratio of ·4 per head.	Existing protected area.	Balance required for complete protection.
Bellary	Adoni	179,000	·014	71,600	2,500	69,100
	Alur	98,000	·016	39,200	1,500	37,700
	Bellary	193,000	·02	77,200	3,800	73,400
	Rayadrug	83,000	·09	33,200	7,200	26,000
Anantapur	Gooty	156,000	·017	62,400	2,600	59,800
	Anantapur	109,000	·06	43,600	6,400	37,200
	Tadpatri	109,000	·04	43,600	4,600	39,000
	Dharmavaram	147,000	·04	58,800	5,400	53,400
Kurnool	Ramallakota	143,000	·025	57,200	3,600	53,600
	Pattikonda	143,000	·02	57,200	2,800	54,400
	Cumbum	116,000	·06	46,400	7,100	38,800
	Markapur	94,000	·13	37,600	12,600	26,000
					Total	567,900 acres.

which approximates to the 575,000 acres mentioned above as coming under 23,000 wells. Part of this area comes under the scheme of the Tungabhadra project, but we are here considering existing conditions only, and proposals must be left out of account.

The selected area lies between 70 and 200 miles from the Kistna dam site, and so is well within the range of practical long distance transmission of Electric Power. In such a scheme the wells would be selected on the principle of the survival of the fittest. The present number of wells within this area is something over 11,000, irrigating 66,000 acres. It is worthy of note that these wells lying within the famine area, and being consequently harder worked than those elsewhere, irrigate on the average 6 acres each, against an average for the whole Presidency of only 3 acres. It is required to increase the number to 23,000 and to raise the average area commanded under each to 25 acres. Of course many of these wells may be expected to fail in a severe drought, and most wells lying on or near water-parting ridges and high ground to fail to come up to the requisite standard; and that, speaking broadly, the successful wells would tend to lie along the lower slopes of the basin of each minor drainage. Important among these wells, would be the "doruvus" or spring wells situated

on the banks of the rivers and streams, tapping the sub-soil flow; and which now irrigate a much larger area than ordinary wells, and which may be expected to irrigate, under systematic pumping, several hundred acres each.

The area of these two blocks may be put at 9,000 square miles and rejecting all high plateaux and concentrating on the lower slopes of each minor basin, the area within which the 23,000 wells would lie, may be put at 2,000 square miles, which would give about 12 wells to each square mile, or 640 acres. As each well would irrigate an average area of 25 acres the total protected area would be 300 acres per mile, or about one-half, which seems to be a convenient proportion. All the existing wells within the selected area would be tested first by a portable engine and pump to ascertain if they could give the requisite discharge of $\frac{1}{10}$ cubic foot per second. Some would no doubt give less, but on the other hand many, especially the "doruvus," may be expected to give more.

This area of 2,000 square miles being distributed among 12 taluks would assuming them to be all of the same area, give about 160 square miles to each taluk. And if a one-mile strip on each side of the river on stream were taken, the collective length of such strip for each taluk would be 80 miles; which would represent the requisite length of main distribution line.

In such a scheme there could be no central distributing station for the reduction of the 30,000 or 40,000 voltage of the main transmission line, to a suitable potential for use at the motors. Instead, there would be a large number of small distributing centres chosen as convenience dictated. Take as a suitable unit a five-mile length of the two-mile strip or an area of ten square miles. This would contain 120 wells requiring ·55 B. H. P. each or 66 B. H.P. in all, for which a small distributing centre containing reducing transformers of 50 kilowatt capacity, would suffice.

In estimating the actual cost of one B. H. P. per annum at the wells, this extra cost in transmission lines and reducing transformers with the cost of attendance must be taken into account. On the other hand in this special scheme we have the storage reservoir of the Kistna Reservoir Project ready to hand, and are in the analogous position of a Steam Engine being supplied with coal free of charge.

In the report of the Committee on the Periyar Utilization of Power, Professor Forbes estimated the cost of supplying one B. H. P. per annum at Madras, distant 350 miles, at the low rate of Rs. 60.

Rs. 60 per H. P. in Madras is an impossible rate. We hope to get as much close to the generating station.

On the other hand the Cauvery Scheme is under agreement after the first five years to supply power at Kolar, 90 miles distant, for Rs. 150. This seems to be a high figure and on a reference being made to him the Chief Electrical Engineer, Mysore, says that it is so, and that a more usual charge would be Rs. 100, which would leave margin for a good profit.

The Gold Fields Company made a splended bargain, and would willingly have paid far more than they do.

Mr. Kennedy in his Technical Paper No. 157 says that in Western America where long distance electrical transmission is coming into vogue for the purpose advocated in this paper, the average cost is Rs. 120 where the supply is continuous. Dr. Bell in his book on Electric Power Transmission gives such low limits as Rs. 20 to Rs. 75, as having been attained in actual practice. Taking our abnormal expenditure on transmission lines, reducing transformers and attendance, as balancing the saving in hydraulic storage, the reservoir being assumed to be already constructed for direct irrigation, it might be thought that to take the high figure of the Cauvery Scheme, *e. g.*, Rs. 150 would be a safe guide; but to be quite on the safe side and to allow for high maintenance charges, let it be put at Rs. 200, making the cost of ·55 H. P. at each well to be Rs. 110.

The capital cost of the power delivered at Kolar in the Cauvery Scheme is Rs. 1,000 per H. P., which the Chief Electrical Engineer pronounces to be very excessive indeed. Adopting this high figure for the charge at the wells we have for 23,000 × ·55 H. P. at Rs. 1,000 the sum of 126½ lakhs of

rupees. Adding to this 23,000 × Rs. 500, being the cost of electric pump and motor at each well, *i.e.*, 115 lakhs, we have 241½ lakhs or say 250 lakhs, as the total capital cost of the scheme; which for 5¾ lakhs of acres comes to less than Rs. 44 per acre. With an average revenue per well of Rs. 250, the gross revenue comes to 57½ lakhs; and with working expenses at Rs. 185 the net profit on 23,000 wells comes to 15 lakhs of rupees, paying 6 per cent. on the capital outlay.

It should be pointed out that in this scheme the load would be nearly or quite constant throughout the 24 hours of the day, and, therefore, admirably adapted for a long distance electrical power scheme.

With regard to this method of protection for tracts liable to famine it may also be pointed out that, *firstly*, it will pay. In their recent report the Irrigation Commission in contemplating large storage schemes for irrigation by direct flow, have been obliged to recommend an expenditure of many crores of rupees which they anticipate will be quite unremunerative. It is possible that some of these schemes might be advantageously replaced by some such scheme as that outlined above. *Secondly*, the present customs of the ryot will not be changed. In all these scarcity tracts he is accustomed to rely on his wells, lifting his water from them by bullock power. If he could afford to buy and maintain more bullocks he would probably dig many more wells, or at any rate get more water from the wells he has. If he is supplied with cheap automatic power he can put down as many more wells as he pleases and use his bullocks only for ploughing and draught. He will not be called upon to substitute irrigation of wet or dry crops by direct flow, for his present methods. His new power also will be quite reliable, whereas in time of famine when most needed, his bullocks either die off altogether or become weakened from want of fodder and so not up to their work. *Thirdly*, no great initial expenditure is required before the success of the scheme can be regarded as certain. A number of wells can be tested all over the selected tract by means of portable pumps, and if the discharge comes up to that anticipated, then the scheme can be confidently carried through, as upon no other points can any doubt arise.

It should be noted in the present instance that all water passing the dam does double duty. In passing the turbines, every cubic foot of water falling 90 feet, does work which serves to raise another cubic foot 30 feet from a well at a distance of perhaps 200 miles. Each cubic foot per second of flow at the dam that enters the turbines may, therefore, be said to irrigate indirectly 250 acres of dry crop before proceeding on its way to the Delta to irrigate another 80 acres of rice.

This particular tract, consisting of parts of the Bellary, Kurnool and Anantapur districts, has been selected as the one in which the benefits conferred by the scheme would be the greatest. Other localities could be selected within, say, a 300-mile radius, where success would be even more probable owing to the greater quantity of sub-soil water. As it stands, however, it provides, if considered feasible, a means of combating chronic famine and scarcity in Bellary, Kurnool and Anantapur, and at a remunerative cost.

It is under consideration to supply Madras and possibly other places, with power for industrial purposes from the Periyar. It would, however, be probably more beneficial to the community at large, to utilize the power which is available there for the extension of irrigation from wells. Madura and Tinnevelly do not suffer much from scarcity, but Coimbatore and Salem do. They also contain more wells than any other districts in Madras, and are within a radius of 200 miles from the Periyar. Also, in using the power for this purpose a break of one or two months in the supply in the hot weather would be immaterial.

The figures in the scheme outlined above are entirely tentative and preliminary. The working out of details has been avoided as far as possible, and effort confined to keeping on the safe side in all estimates of cost, etc. The best authorities available have been consulted throughout, and on this account the scheme may be thought worthy of fuller enquiry.

Irrigation of sugarcane by pumping in Behar 1904.

The cultivation of sugarcane in Behar has developed largely during the last few years. Improved kinds of cane have been imported and much attention given to manuring, irrigation, and tilling of the soil; it has already been shown that the output of sugar from the ordinary cane is increased by irrigation, and that imported large cane cannot with advantage be grown without it. Only a small portion of Behar is commanded by canals, consequently irrigation by pumping has to be resorted to.

The Government of Bengal is anxious to assist the planting community by conducting experiments in order to ascertain the best method of growing sugarcane successfully, and as regards the cost of irrigation, and with this object conducted experiments by pumping from the Bur Gandak river at Otter, with a 10″ Invincible Centrifugal Pump and a 10 H. P. Engine. Unfortunately a start was made late in the season, and heavy rain in May stopped the demand for water. The experiment will be continued in 1905 with a larger plant, *viz.*, a 15″ Centrifugal Pump and a 16 H. P. engine.

The information obtained from these experiments is:—That a 10″ pump with a lift of 27 feet and lead of 4,000 feet, running at 650 revolutions a minute, can throw 3½ to 4 cubic feet of water per second; *i.e.*, 1,300 to 1,500 gallons a minute, and irrigate at the rate of ⅓ acre per hour. About 5 inches to 7 inches average depth of water was utilized on the fields which were trenched, 3 inches to 5 inches was lost in the new channels. The engine used two maunds of coal per hour at seven annas a maund; labour for working the engine was somewhat excessive, costing Rs. 1-10-0 per day of 11 hours, the total cost of coal, oil, labour, etc., came to Rs. 3-6-0 per acre for one watering.

Irrigation was carried on at several concerns by steam pumping on their own account, and the proprietors have kindly allowed the results to be published for general information.

It would possibly pay Planters to use Hornsby's (Grantham) oil engine, or some equally good one, instead of a steam engine.

One interesting question to decide is, the number of waterings that may with advantage be given to sugarcane; in Egypt the crop is watered once every 10 days for 75 days, or an average of 7½ waterings.

Statement showing particulars of sugarcane irrigation by pumping at various Factories in the Gandak circle during the year 1904.

Concern.	Size and kind of Pump.	H. P. of engine.	Date.	Number of hours of pumping.	Rain-fall in inches.	AREA IRRIGATED, ACRES. DETAIL OF IRRIGATION. 1st.	2nd.	3rd.	Total.	Cost of coal, oil, etc.	Cost of engine driver and labour.	Lift (Height to which water was lifted.)	Lead (Average distance of fields from pump.)	Total cost of channels, etc.	Cost of irrigation per acre. Single watering column (11 + 12)/10	REMARKS.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
						Acre.	Acre.	Acre.	Acre.	Rs. A. P.	Rs. A. P.	Ft.	Ft.	Rs. A. P.	Rs. A. P.	
Government figures at Otter.	10" Gwynne's invincible centrifugal Pump.	10 H. P.	25th April 1904 to 7th May 1904.	141	1·50	46·00	...	...	46·00	134 0 0	21 0 0	26·53	4,000	*250 0 0	3 6 0	There was considerable loss due to percolation in the new channel. * Average cost of earthwork per mile.
Seeraha . .	Feed Pump . .	10 H. P.	10th March 1904 to 20th April 1904	No record	Not shown.	80·30	18·75	...	99·05	310 0 6	40 10 0	25	1,055	118 6 9	3 8 4	
„ . .	Centrifugal Pump	12 H. P.	12th March 1904 to 12th April 1904.	Do.	Do.	148·55	35·55	...	182·10	468 7 9	70 15 9	10	3,579	20 2 6	2 15 4	
„ . .	Worthington Pump	Not known.	12th April 1904 to 5th May 1904.	Do.	Do.	76·53	4·00	...	80·53	319 11 4	52 9 6	25	4,932	53 8 0	4 10 0	
„ . .	Not known . .	Do.	2nd to 20th April 1904.	Do.	Do.	79·15	...	...	79·15	748 8 9	50 14 9	10½	4,000	50 9 0	10 2 0	
Total .						384·53	56·33	...	440·83	1,846 12 4	215 2 0	×	×	242 10 3	4 10 9	
Tandpur .	7" Centrifugal Pump	10 H. P.	... ...	...	No record.	75	...	...	75	124 4 0	68 4 0	16	Within radius of 500 yards.	...	2 9 0	
Moosheri .	Centrifugal Pump 9" and 7".	12 and 10 H. P.	... ...	...	Do.	60	...	...	60	145 0 0	50 0 0	15 to 20	¼ to 1 mile.	90 0 0	3 4 0	
Total .						135	...	...	135	269 4 0	118 4 0	×	×	90 0 0	2 14 0	
Barhogia .	4" Special steam Pump.	80 H. P.	14th February 1904 to 10th June 1904.	414	3·93	34·00	9·63	8·37	52·00	94 13 0	19 5 0	11	¼ mile	32 4 3	2 8 0	The water from the Pump having been available only spasmodically, that is to say, when the pump was not being used in connection with crushing cane, has caused the average cost to be so high. † Includes constructing masonry drain, carrying water under ground 240 feet and distributary channel.
„ . .	6" Gwynne's centrifugal Pump.	8 H. P.	2nd March 1904 to 7th June 1904.	342	3·93	62·30	15·31	5·92	83·53	89 12 0	35 7 9	21	¼ mile	25 0 0	1 11 9	
„ . .	8" Gwynne's centrifugal Pump.	12 H. P.	16th February 1904 to 18th June 1904	477	3·93	102·55	36·48	10·37	149·40	233 10 0	57 12 9	31	½ mile	†377 5 0	1 15 3	
Total .				1,233	3·93	198·85	61·42	24·66	284·93	418 3 0	112 9 6	×	×	434 9 3	1 13 10	
Otter . .	6" & 8" Centrifugal 8" Special Pump .	12 H. P.	3rd March 1904 to 28th May 1904.	212	Nil.	212·25	26·25	...	238·50	690 3 3	256 7 7	26	½ mile	89 2 0	3 15 6	
Total .				212	...	212·25	26·25	...	238·50	690 3 3	256 7 7	26	½ mile	89 2 0	3 15 6	
Ramcola . .	6" Centrifugal Pump.	10 H. P.	20 days in March and May 1904.	...	2·6	40·00	3·00	...	43·00	100 0 0	40 0 0	28	½ mile	5 0 0	3 4 0	
Total .						40·00	3·00	...	43·00	100 0 0	40 0 0	28	½ mile	5 0 0	3 4 0	

PAPER No. 23.

The Jamrao Canal.

As originally constructed, the Jamrao Canal was 125′ bed width at the head and the depth of water was 8′. This was designed to give a discharge of 3,200 cusecs with a velocity of 2·5′ per second.

The first 13 miles of the Canal are through sand, and from 7 to 13 miles a shallow depression has to be crossed. It was considered advisable to introduce a Fall at 7¼ miles to keep the Canal in cutting. The fall is of the notch type, and provides for a fall of 8′ in the bed of the Canal and of 6′ in the water surface. Below the Fall, the bed width was reduced to 104′, and the depth of water was increased to 10′. From the head to 7¼ miles, the bed fall was designed to be 1 in 5,000′ and from 7¼ miles to the tail 6″ per mile; but, for reasons to be mentioned later, these bed falls have changed considerably in the first 30 miles.

The total length of the main Canal is 117 miles, and regulators have been provided at approximately every 10th mile. The west branch is 63 miles long, and there are 408 miles of minor canals. With the exception of the upper part, the Canal has worked most satisfactorily. There has been no scour, and no silt has been deposited, except at the extreme tail.

As stated above, the upper part of the Canal is through fine sand and the bed fall is 1 in 5,000. As soon as water was raised to 8′, the surface velocity increased to over 4′ per second, the sides began to scour, and, as the average depth of cutting was 15′, they rapidly fell into the Canal and almost filled it up as the amount of sand falling in was considerably greater than the sectional area gained in width. Increasing the depth of water at the head merely added to the scour and did not increase the discharge.

The remedy adopted was to allow the sides to scour away until a width of a little over 200′ at the water surface had been reached, and then brushwood groynes were made to limit the width of the channel to 200′. These groynes were first made 100′ apart and varied in length according to the amount of scour which had taken place at any particular point. In addition to the groynes, trial was made of continuous stakes and brushwood protection along the sides of the Canal as soon as the width had reached 200′; but this was not successful.

As originally constructed, the groynes put a stop to all further scour of the sides; but they were too far apart to collect silt properly between them, and they were so weakly made that, as soon as an attempt was made to get full supply into the Canal, the ends were carried away.

It was therefore decided to construct groynes at every 50′ and to strengthen the outer ends by thicker stakes driven deeper into the bed of the Canal. The result of these measures became apparent almost at once. In February 1901, with 8′ of water at the head and the original system of groynes, the average depth of water above the falls was 3′. In March, the Canal was closed, new groynes were made, and the old ones strengthened. The Canal was reopened on the 1st May, and by the end of the month, with 8·5′ of water at the head, the average depth above the falls was 4·5′. This continued to increase till, in October, the average depth was 5·75′. In April 1902, when the Canal was closed, it was found that the bed had, as a whole, scoured considerably, and there was a deep channel winding from side to side. The groynes were still further strengthened and improved, and when the Canal was reopened in May, the depth of water at the head was raised to 8·75′. The bed continued to scour, and in November 1902, the average depth of water above the Falls was 6·0′. In April 1903, when the Canal was closed, it was seen that the section had improved very much. It was nearly uniform from the head to the Falls, the deep winding channel had disappeared, and the bed was level from side to side. During the closure, necessary repairs to the groynes were carried out, and, when the Canal was reopened, it was decided to raise the water at the head to 10′, if necessary, to get the full designed discharge of 3,200 cusecs. This was obtained with 9·5′. The groynes and silt between them

stood this perfectly, and the bed scoured a few inches deeper. At present, the average depth of water above the Falls is just over 7·0′, *i.e.*, whereas in 1901 there was an average depth of 5′ of sand above the designed bed level, there is now only 2·3′.

The groynes are constructed of double rows of stakes, about 3″ in diameter. The rows of stakes are 5′ apart and the stakes in each row are 1′ apart. They are driven as deeply as possible into the bed of the Canal, and brushwood is interwoven between the stakes. The space between the two rows of stakes is filled with brushwood tightly packed.

The groynes are only of a temporary nature, and require constant supervision and repairs. Two gangs of men are permanently employed in attending to them, repacking brushwood and driving in stakes which have worked loose. It was hoped that the silt collected between the groynes would be stiff enough to resist scour and that it would not be necessary to maintain them for more than a year or two; but unfortunately this had not proved to be the case, and apparently they will have to be maintained permanently.

The sand scoured from the bed in the first 7 miles has all been carried over the Falls and has filled up the Canal below to such an extent that the water level now is 4′ higher than originally designed.

This has not caused any inconvenience beyond having to raise the banks on both sides from 9½ to 12½ miles. In 1901, all this sand was confined to the 8th and 9th miles, but it is gradually extending down the Canal, and the new bed slope just below the Falls is flattening out. In 1902, the sand had reached the 17th mile and in 1903 the 20th mile. This year it had just reached the 30th mile. This has naturally raised the water level all along; but as the banks are still from 4′ to 7′ above water level, it has not been necessary to do anything. Except in one or two very exceptional cases, no irrigation is allowed direct from the main Canal, and the various villages draw their supply from minors. The total length of these minors is 408 miles, and practically all of them take off the main Canal just above the regulators. The water in the minors can thus be kept at a fixed level, and the supply passed down them is constant. If for any reason there is not enough water in the main Canal to fill all the minors, they are worked by rotation. Some are closed entirely, and the water level in the reach of the main Canal in which the minors are open is raised to full supply level by use of the regulator at the end of the reach. The water in the minors which are working is thus always at full supply level. The bed falls of the minors vary from 4½″ to 11″ per mile, and the cross sections are reduced at suitable places, so that the discharge always corresponds to the area to be irrigated.

All the minors silt to a certain extent; but this is very small, when compared to any of the inundation canals in Sind. The maximum depth of silt deposited does not exceed 3·0′, and this only in a few cases at the tails of some of the minors. The average depth of silt is about 1·5′, and this has to be removed every year.

Head Works.

The Head Works of the Jamrao Canal are situated on the right bank of the Eastern Nara River, about 100 miles south of Rohri, where this river takes off the Indus. They consist of right and left flank embankments to confine the river to the desired channel, training banks to direct flood water on to the weir and off the flank embankments, a head regulator, under-sluices and a masonry weir across the river. The general arrangement of these works is shown by Fig. No. 1 (Plate 39.)

The right flank embankment is 3½ miles in length. It begins at the upstream wing wall of the head regulator and terminates at the sandhills which form the western boundary of the Nara Valley. The left flank embankment is about 1 mile long. It begins at the left abutment of the weir and is continued to a high sandhill called the Sihori Bhit.

Both these banks are constructed of sand covered with 1 foot of *paka* earth. The top width is 15′ and the side slopes 2 to 1, except at one place,

where a deep channel is crossed. Here, the top width was increased to 20′ and the front slope to 6 to 1 and the back slope to 4 to 1.

The top of the banks is at R. L. 120·00 or 10′ above maximum estimated flood level. In the left flank embankment, a gap 300′ in length has been left with the top at R. L. 114·00. This gap is meant to act as a safety valve in case a larger flood than that estimated should come down the river. In such a case instead of injuring the head works, the bank would be overtopped and breached at the gap, and the flood would pass off harmlessly into the desert.

The training banks are four in number, and are all of the same section, *viz.*, 15′ top width, side slopes of 2 to 1 and top at R. L. 121·00. These banks are also constructed of sand covered with 1′ of *paka* earth, but in addition the sides exposed to the stream are protected by dry brick pitching 1′—9″ thick resting on a concrete toe, beyond which an apron of rubble stone 30′ broad and 3′ thick is laid. The section of these banks is shown by Fig. No. 2.

The dry brick pitching consists of 6″ broken brick on which a brick is laid flat, and on this brick on end 12″ deep. The pitching concrete toe and rubble apron are carried completely round the ends of the banks and for 50″ along the back slope.

Both the upstream banks and the left downstream bank are 700′ long. The right downstream training bank differs from the others, as it was considered necessary to divert the main stream of the river away from the Canal. This bank is therefore carried right across the original bed of the river and on to the opposite bank. The length is over 2,000′ and it is slightly concave to the stream. So far, scour has only occurred at the end of the left upstream bank. In the inundation of 1902, the main stream of the river set directly on to the end of this bank, and a hole about 100′ long and 10′ deep was scoured out along the edge of the rubble apron. This apron acted exactly as it was hoped it would. About half of the breadth fell into the hole, thus pitching the side, and there was no further scour. Before the next inundation, two brushwood groynes were constructed on the bank of the river to turn the current off the end of the bank, and the scour hole has now silted up to the level of the river bed.

The Head Regulator consists of 6 spans of 25′ as shown by Fig. No. 3. The wing walls, abutments and piers are founded on wells 10′×8′ and 10′ deep, the tops of these wells being from 8′ to 10′ below spring water level. The wells are spaced 8″ apart, and the openings between are closed by sheet piles 6″ thick and of the same length as the depth of the wells. The wells are filled with concrete of broken brick and hydraulic lime mortar.

The piers are 30′ long and 4′ thick. The pavement between them is 4′ deep.

At the downstream end of the wing walls, a row of wells 10′ deep was sunk as a curtain wall. The space enclosed by the regulator, wing walls and curtain wall is paved with 1 foot of brick masonry resting on 2 feet of concrete, below which there is 6″ of broken brick.

The bed and sides of the Canal below the curtain wall are also paved for a length of 173′. The first 74′ is of brick and lime masonry 2′ thick, then 24′ of concrete blocks, on 6″ of broken brick, each block being 2′×2′×1′. Below the concrete blocks, the paving is 1′ of dry brick on end on 6″ of broken brick. The regulation of the water is effected by means of rolled steel beams, placed horizontally and working in cast iron grooves built into the cutwaters of the piers. The object of these horizontal beams is to keep as much sand and heavy silt as possible out of the Canal by taking water from the surface of the river only. These beams are of three sections, but are all made up to 6 inches in depth by teak planks bolted to the web.

D-shaped handles are also bolted to the web at either end of the beams to take the hooks of the lifting chains. For raising and lowering these beams, the arrangements are as follows. Masonry walls are built over the piers and cutwaters as shown on Plate No. 40, and on these two lines of rolled beams are

laid—one over the outer end of the cutwater and the other on the roadway end of the wall. To these beams are fixed angle irons at about 1 foot from the supporting walls. Wheeled runners are placed on these L irons and to the runners differential pulleys are hooked. The beams for closing the openings are stacked in the roadway, and, when required, one is lifted by a pair of pulleys, run out over the grooves in the cutwaters and lowered into place.

This system is not very satisfactory. In the first place, the differential pulleys work very slowly and a great deal of time is unnecessarily wasted, especially when raising or lowering the hooks without any weight on them. A pair of ordinary pulleys has been tried instead of the one differential pulley, and it was found that these were quite powerful enough, and they worked at more than double the speed of the differential pulleys.

A more serious objection to the differential pulleys is that after about a year's use the links of the chains stretch a little and no longer fit into the grooves in the circumferences of the driving wheel. The chains then jam and the links have to be hammered into the correct shape again. The amount of stretch is not enough to be detected by the eye, but it is quite enough to render the pulley useless for the time being, and a man with somewhat more skill than the ordinary blacksmith is required to put the matter right.

The under-sluices, Plate No. 41, are 7 spans of 20′ with sill level at the same level as the bed of the Canal. They are almost at right angles to the face of the Head Regulator and are separated from the weir, which is in line with them, by groyne walls both up and down stream. The whole area between these groyne walls and the face and wing walls of the Head Regulator is paved with masonry, the upstream side with a thickness of 1′ of brick on end on 1′ of concrete and the downstream side for a distance of 45′ with 4′ of brick-masonry. Between the piers of the under-sluices, the pavement is 5′ thick, the upper 18″ being ashlar and the lower 3′—6″ brick and lime masonry. Between the upstream end of the groyne wall and the Head Regulator, a curtain of wells was sunk and an apron of concrete blocks 4′×4′×2″ was laid for a length of 20′.

At the downstream end of the pavement, another curtain of wells was sunk, and below these concrete blocks were laid for a length of 140′. The under-sluices and groyne walls are founded on wells similar to those in the Head Regulator.

Each archway is closed by 3 horizontal steel gates, the top of the upper gate being 3′ above full supply level in the Canal. These gates are raised and lowered by crab winches, which work quite satisfactorily, but painfully slowly.

The weir has a clear waterway of 1,250′ divided into 5 bays of 250′ by masonry towers. The crest was fixed at R. L. 102·00 or 1 foot below full supply level in the Canal, and in order to assist the under-sluices in drawing the main stream of the river towards the Head Regulator the two bays next to the under-sluices were made with the crest 2 feet lower than the rest of the weir. To prevent currents crossing the weir obliquely and so scouring a channel along the upper edge, the upper and lower weir were separated by a groyne wall.

The central portion of the weir is made heavy enough to withstand the maximum upward pressure to which it could be subjected, *viz.*, that due to a head of 8′ of water, and this could only occur when there is no water at all below the weir. The upstream part of the weir is a brick masonry platform with a small curtain wall, 3′ deep at the end. In front of this curtain wall there is a line of sheet piling 7′ deep, and in front of the piles a rubble stone apron 15′ broad and 3′ deep. The downstream side slopes 1 in 10, is 74′ broad and 3′ thick. Half way down the slope, there is a small, stop wall, 2′ deep, to check any accumulated creep of water. At the downstream edge of the weir, there is a curtain of 10′ wells and below these 40′ of concrete block pitching. The cross section of the weir is shown on Plate No. 42. Falling shutters have been fixed on the crest of the weir to raise the level of water when necessary. In the original design, 6′ shutters were provided for the lower weir and 4′ ones for the upper; but during construction, it was found that 4′ shutters would probably be sufficient on the lower weir and 2′ ones on the upper. As all the

shutters were the same width and the hinges of the same pattern, the 4′ shutters intended for the upper weir were fixed on the lower and temporary 2′ wooden shutters on the upper weir.

After three seasons' trial, it was found that this arrangement was quite successful, and the temporary wooden shutters have now been replaced by iron ones.

The 4′ shutters are copies of those on the Khanki weir illustrated on plate No. 5; but the 2′ ones are plain sheet iron, $\frac{1}{4}''$ thick, stiffened by angle irons at the edges.

The shutters are raised and lowered by hand. Two men can raise a 4′ shutter with $2\frac{1}{2}'$ to 3′ of water passing over the weir, and the only difficulty occurs in raising the last two or three shutters in a span. In these cases, four men are required—two to lift and two to pull on a rope from the upstream side.

On the two abutments and on the tops of the masonry towers separating the bays of the weir, iron lattice work towers have been fixed. These carry a steel wire rope, on which wheeled carriers run. This rope tramway was supposed to be used for crossing the river when in flood. It cost a lot of money, and has never been of the slightest use.

Distribution of water.

The system of distribution of water on the Jamrao Canal is modelled on that developed on the Chenab Canal. All the land on the Canal has been divided up into squares of very nearly 16 acres each, the side of a square being 840′ long. The whole country has also been levelled, levels being taken along the boundaries of the squares at intervals of 420′. From these levels, contour lines were laid down on the plans, and then the natural drainage lines and village boundaries. The standard size of a village was fixed at 2,000 acres, and the maximum length of watercourses at 3 miles; but these dimensions have been exceeded in very many cases. This had to be done because all the minors, except one, had been excavated before it was decided to adopt the Chenab system. The result is that they are not aligned in the most suitable manner to irrigate all the land they ought to command, and the water has to be most carefully distributed to reach the tails of the long watercourses and all the land to be irrigated.

Each village is provided with one or more masonry outlets from the minor and a system of watercourses to carry the water to every field in the village. The watercourses are aligned along the sides of the squares or diagonally through them, and have a bed fall of 1 in 8,400.

The amount of water allowed is one cusec for every 300 gross acres, so that a village of 2,000 acres would get 6·66 cusecs.

The outlets are small brick culverts, extending under the bank and the service road behind it. They are all of one type, and only three sizes were built, *viz.*, of 1′, $1\frac{1}{2}'$, and 2′ span. It was estimated that the velocity through the outlet would be 2·5 feet per second, but this is too low, as the head is rarely less than 6″, and this gives a velocity of 3·55 feet per second. However, the size of all the outlets was calculated for a velocity of 2·5 feet per second, so that for a village of 2,000 acres the sectional area of opening necessary is $2' \times 1{\cdot}33'$. For this, a 2′ span outlet was used, and the exact size of opening required was obtained by building a stone slab into the arch and abutments with the lower edge at the calculated height above the sill of the outlet.

The outlets are 27′ long, and it was feared that they might silt up and that it would be troublesome to clear them; but this has not occurred. Except in very special cases, there are no sluice gates to these outlets: their size having been limited to the area to be irrigated, they are always open.

The minors also are always run at full supply level, or if there is not sufficient water in the main Canal for all, some of them are closed entirely. This is the essential point of the system, *viz.*, that the minors always run with a definite known supply, and with the water at a fixed known level, or they are empty.

The distribution of the water amongst the various cultivators is carried out by Canal Assistants and Abdars. There is a Canal Assistant in each sub-division and from 6 to 8 Abdars under each Canal Assistant.

Each Abdar has 5 villages, or from 10 to 12 thousand acres, in his charge. The duties of an Abdar are to show the cultivators from which watercourse and in what way they are to take water, to inspect each village in his charge in detail at least once a week, to submit a weekly statement of the area cultivated in his beat, to report all cases of waste or unauthorised use of water, and to report if there is any deficiency of water in any of his villages.

The Canal Assistants supervise the work of the Abdars, check all reports of waste, etc., of water before submitting them to the Sub-Divisional Officers, and, if rotation is necessary, they make the preliminary enquiries.

When a complaint of deficiency is received, the outlet is first inspected and then the watercourses. The depth of water in the latter is measured at various places to see the amount of silt in them, as many complaints are due to want of clearance and not to actual deficiency of water.

If the outlet and watercourses are found to be in proper order, it is plain that the complaints must be due to a larger area having been cultivated than was designed to be irrigated in one season, and rotation must be fixed to give every one his share of water. This is done by allowing each branch watercourse in turn to take water for a certain number of hours, according to the area cultivated on it. The people are given written notice of the rotation fixed; but it does not necessarily follow that they will adhere to it. Those who break the rotation are fined or prosecuted, according to the gravity of the offence.

When fixing the amount of water to be allowed per acre, it was estimated that $\frac{1}{3}$rd of the total culturable area would be cultivated in any one year, as this is the universal custom on all the other canals in Sind. Of this $\frac{1}{3}$rd, it was estimated that $\frac{8}{13}$ths would be cultivated in the Kharif season and $\frac{5}{13}$ths in the Rabi season, or very approximately $\frac{1}{5}$th of the total area in Kharif and $\frac{1}{8}$th in Rabi; but in the Kharif season this estimated area is always greatly exceeded, in most villages a full $\frac{1}{3}$rd and in some a $\frac{1}{2}$ being cultivated. Hence, the necessity for rotation on the branch watercourses.

There are other reasons which render rotation necessary. One is the large size of some of the villages and the consequent great length of the watercourses. In these, without rotation, water never reached the tails of the lower watercourses. It was a very troublesome matter fixing a fair rotation in these large villages, and to render it easier fire-clay drain pipes, varying from 3″ to 12″ in diameter, were this year placed in the heads of all branch watercourses. These have acted more satisfactorily than anticipated, and it has not been found necessary to have rotation in any of the villages in which these pipes have been fixed. The upper branch watercourses not being able to draw off more than their fair share of water, the main watercourse works as it was intended to, and the water is distributed all over the village instead of being used up at the head.

Other difficulties encountered in the distribution of water are due to the character of the water-supply the people previously had. Prior to the Jamrao, all the cultivation was on the extreme tails of old inundation canals taking direct off the Indus and without any head regulators. Very few of these canals were even in fair order, and the supply was most uncertain. Sometimes, it was fair; but generally it was poor, and at times the water would dry up entirely for 10 or 12 days at a time. The cultivators, therefore, took water at every opportunity, without regard to the wants of their neighbours or those lower down the canal. When the Jamrao was opened, they could not realise that a steady and certain supply had been provided, and for the first three

seasons they gave great trouble by behaving just as they had been accustomed to do on the old canals. They put *bands* across minors and watercourses to raise the water level, and those who had land at the head of a watercourse would not let any water pass their land till they had taken all they wanted. They have now learned that the supply can be depended on, and they no longer *band* the minors; but they frequently do so on the watercourses and thus upset the rotation fixed.

Again, on the old canals, there were no restrictions as to the amount of water a man was to take, and practically no notice was taken of waste of water. Provided the head of an existing watercourse were not enlarged, any number of channels could be taken off this watercourse. As none of these old watercourses had masonry heads, this meant that a cultivator could take just as much water as he pleased from the Canal.

When the new system was introduced, they greatly resented the strict rules, especially those forbidding the excavation of watercourses wherever they considered they were necessary or more convenient than those made by the Public Works Department. They still look upon this as a hardship, and excavate such watercourses, if they think they will escape detection; but as they never do, this practice is gradually dying out. The Jamrao is the only canal in Sind on which the Chenab system has been introduced and is in working order, and there can be no doubt of the success of this system.

As already stated, water is only guaranteed for ⅓th of the total culturable area in the Kharif season; but in 1901, which was a year of record bad inundation in Sind and one in which the area cultivated on all the canals, except one, fell off greatly, the guaranteed area was exceeded in the Northern District of the Jamrao Canal by 20,000 acres and the remissions of revenue granted owing to deficiency of water were under Rs. 400. Last year, the guaranteed area was exceeded in the same district by 23,000 acres, and there were no remissions on account of deficiency of water.

The culturable area commanded by the Canal is 822,000 acres, and it was anticipated that 268,000 acres would be cultivated annually when the project was fully developed and all the land had been given out for cultivation. In 1902-03, the latest year for which figures are available, the area cultivated was 269,358 acres, and a considerable area still remained to be given out.

PAPER No. 24.

River Training in Tanjore.

Natural origin of the Cauvery Delta System.—The so-called "rivers" of the Cauvery Delta are for the most part the natural ramifications of the main river characteristic of all deltas, but have been used as sources of irrigation from times far anterior to the advent of British rule and scientific engineering.

They have, however, been gradually brought under régime and control by diversions and embankments and by the conversions of solid dams into regulators, the improvement of old regulators and the construction of new ones—this especially during the past decade—so that at the present day they form a system of deltaic canals under complete control.

Effect of natural origin.—The result of the natural origin of the rivers is that there is no uniformity in bed width or width between embankments, while their courses are, as a rule, irregular and in some cases little less than tortuous; and when to these characteristics are added a comparatively rapid fall corresponding with that of the country and a consequently high velocity, it will be understood that careful and constant training is very necessary to keep the rivers within bounds, a necessity hardly felt at all in the case of properly designed artificial canals.

Scope of paper.—Now, the main principles of river training are more or less universal and it would therefore be out of place to dilate on them in this paper, but the methods by which these principles are given effect to in the Cauvery Delta include some which are peculiar to it, and these it is proposed to describe.

Tanjore rivers and "padugais" described.—In most of the Tanjore rivers there is a strip of land on both sides between bank and margin, which may vary, owing to the absence of any artificial alignment, from a few feet of berm to several hundred yards of dense plantain garden.

These strips of land, or "*padugais*" as they are called, are very fertile and much sought after and, with the exception of isolated plots that the Public Works Department have managed to retain, are held by the ryots, who grow valuable crops of tobacco and plantains on them.

Scope of conservancy work in Tanjore.—Any wholesale straight cutting of a river course is thus out of the question as the cost of land acquisition would be prohibitive, and all that can be done is to prevent the aggravation of recognised bends, to protect margins and banks from local erosion, and to reclaim ground where preventive or protective measures have failed.

Conservancy—Plantations in general.—In the Tanjore District the primary means by which the oscillation of a river is kept within bounds and ground reclaimed is the planting up of the bed and margin at suitable points in such a way as to check the velocity of the water and to cause the formation of accretions of silt.

Nánal plantations.—The plant universally used for this purpose is "*nánal*" or "*darbá*" (*ochalandra rheedii*), a hardy and quick-growing reed that attains a height of 10 feet under favourable conditions.

Nánal planting is not entirely confined to the Cauvery Delta, so need not be described in great detail here. It will be sufficient to say that wisps of nánal from 2 to 3 feet in length are dibbled in rows into the portions of the bed extending beyond the alignment required, alternate and opposite bays in the river margin being dealt with simultaneously. The operation is carried out on the subsidence of freshets, the falling water being closely followed so that the nánal stalks may have the benefit of the moisture and take root before the bed dries up.

The required alignment is not worked up to all at once, but operations are spread over several seasons, the edge of a plantation being extended in

gradually flattening curves until the required object has been attained (see Plate 44, Fig. 1).

This method of reclamation and realignment is much cheaper than any other treatment, running as it does to only 4 or 5 annas per 1,000 square feet when materials are close at hand, but at the same time it requires a pre-arranged plan of operations and constant nursing, conditions which are not favoured by the frequent changes in supervising staff and the extensive charges of the present day.

Other plants employed.—Other plants which are employed in training operations in the Tanjore District are "wild cane" (*ochalandra setigera*), "korukkai" (*ochalandra stridula*) and "nir nochi" (*vitex trifolia*).

Wild cane—korukkai.—The first of these is dibbled into the margins and slopes of banks and thrives luxuriantly throughout the delta. The second, which is also a species of cane, is now being used in the same way. These two grow best in alluvial soil and will not stand much submersion, requiring that several feet of it should always be above water. Nánal, on the other hand, thrives in pure sand and is uninjured by submersion so long as the heads are just clear.

"Nir nochi."—"Nir nochi" is a kind of willow with a very tough root and stem, and will take root in several feet of water. It is of great value in securing margins and protecting them from being undermined. It is also used as a reinforcement to nánal, a line of it being formed along the outer edge of a plantation with parallels at intervals between this and the margin.

Silt fences.—When the current in a bay is too strong for the young nánal to stand, even with the addition of lines of "nir nochi," the plantation is reinforced with what are known as "*silt fences*" which have the effect of checking the current and encouraging the deposition of silt.

Component parts of silt fences.—In the composition of these silt fences, there are two elements which in various combinations constitute the distinctive feature of river conservancy in the Cauvery Delta, to which they are peculiar. These are the *horse* and the "*modassel.*"

The horse.—The horse is a raking strut made up of the trunk of a tree cocoanut, palmyrah or small jungle tree or of a stout bamboo, according to the size required, supported at the butt end by means of two straddling legs of casuarina, junglewood or bamboo (see Plate 45).

"Modassel" work.—"Modassel" work is made up of, *firstly*, a line of piles or stakes, usually of casuarina or bamboo accordings to the size required, connected and formed into a compact frame-work by means of bamboo wallings; and, *secondly*, a permeable fence of bamboo twigs and thorns 4 to 6 inches in thickness sandwiched between two trellised frames of split bamboo.

The single silt fence.—The simplest form of silt fence used to reinforce a nánal plantation is constructed as follows (see Plate II, Figs. 2, 3 and 4).

A line of horses at an angle of between 30° and 45° downstream is fixed in the bed of the river, with butts upstream and legs planted firmly in a continuous vertical plane and driven about 3 feet into the bed, leaving the butt ends of the horses from 3 to 4 feet above it with the tail ends embedded about the same depth and secured with stout pickets. Bamboo wallings are then nailed to the row of legs at intervals of 2 or 3 feet, one being placed immediately over the butts of the horses and one at bed level. Using this frame-work as scaffolding a row of stakes or piles is driven about 3 feet into the bed against the wallings, to which they are nailed, leaving a height of 4 or 5 feet above bed. These stakes are driven about 2 feet apart and are connected along the top by walling pieces at back and front.

The "thorn fence."—"*Thorn fences*" or "*tatties*" are next prepared in handy lengths, by spreading a 4 to 6 inch layer of bamboo twigs on a trellis of split bamboos nailed together from 12 to 18 inches apart, laying over it a simiar trellis with a 6 to 9-inch mesh and lashing the whole securely together. These "tatties" are then up-ended and lashed to the front of the row of stakes

to their full height. The total height of a silt fence is usually determined by that of the adjacent padugai, without reference to water levels.

Similar silt fences are erected in parallel lines across the plantation at intervals of from $1\frac{1}{2}$ times to twice their length. They are not intended to divert the current though they may do so in a minor degree when not submerged but to check it, assist in the deposition of silt, and generally strengthen the plantation (see Plate 44, Fig. 1).

Formerly all connections were made with coir rope, but French nails are now used as far as possible, as the lashings either rot very quickly or are stolen. It is intended to try galvanized wire, where nails cannot be used, if not too costly.

The double silt fence.—When a more solid obstacle is required than that afforded by the structure just described, a "*double silt fence*" is formed by the addition of a row of stakes 3 or 4 feet in rear, connected up with wallings, as in the front row, and tied to the latter with cross pieces (see Plate 44, Fig. 4). The space thus enclosed is then filled with bundles of brushwood.

Closing "kalagams" with silt fences.—This latter form of silt fence is more particularly used in the blocking up of an undesirable channel or "*kalagam*" as it is called in Tanjore. Several of these fences are thrown across the channel and the intervening spaces planted up with náaal, at once if the depth of water permits or as soon as the fences have caused the deposition of a sufficient depth of silt.

Another method.—A special device was adopted with success last year in a case where the water was too deep for planting. Two parallel single silt fences, 30 feet apart, were carried across the kalagam from "padugai" to shoal and the outer ends connected with a similar fence (see Plate 44, Fig. 5). This enclosure was then filled in with sand up to water level and planted up thickly with náal.

Groynes.—When it is desired to divert the current, some form of groyne is of course required, and the form used throughout the Cauvery Delta is again a combination of horse and modassel work.

The single horse groyne.—The simplest form of "horse groyne" is exactly similar to the single silt fence already described, with this difference, that the top is always kept clear of M. F. L. by a foot or two and that, on account of the greater height, the tails of the horses have to be embedded in the bank or padugai and not in the bed of the river.

The upstream face of the groyne is placed at an angle of 30° to 45° and the downstream face at right angles to it. The length of the former is thus limited by that of the outer horse, the practicable maximum for which is about 30 feet. The height may run up to 18 or 20 feet in the Coleroon, but ranges from 6 to 12 in the irrigation rivers. In the single groyne the lower end is closed by means of a row of piles and wallings and the triangular space thus enclosed is filled in with bundles of brushwood up to M. F. L. (see Plate 45, Fig. 5).

Double horse groyne.—A "*double groyne*" is formed in the same way as the double silt fence by the addition of a row of piles and wallings connected to the outer row with cross ties. The lower end is usually closed by a single row of piles and wallings, but in some cases, more especially in the Coleroon, the double row has been carried all round and tatties fixed to the downstream face also (see Plate 45, Figs. 1, 2, and 3).

Instead of using double groynes single groynes can be strengthened and solidified by driving a cross row of piles against the trunk of each horse (see Plate 45, Fig. 4), the space enclosed by the groyne being thus divided into several compartments, each of which is filled with bundles of brushwood. This is the more recent practice.

The modassel.—When it is necessary to put an immediate check to the erosion of a bank or margin, recourse is had to "modassel" work in front of and parallel to the erosion, supported by horses resting on the bank or margin (see Plate 46, Figs. 1 to 5). The details of construction are practically the same as for groynes with the same variations in form.

"Modassels," as they are called, are usually kept above M. F. L., but this is not so essential as in the case of groynes as they are not liable to set up scouring action if overtopped. The ends are always brought in to the bank or margin and enclosed (see Plate 46, Fig. 5), so that the velocity of water passing behind may be checked. With the same object, and to assist in the deposition of silt, the space enclosed is filled in with bundles of brushwood, all accretions being planted up at the first opportunity.

The height of modassels varies from 6 feet in the irrigation rivers to 20 feet in the Coleroon, and in same cases those exceeding 12 feet in height are supported by a lower tier of short horses, the horses of the lower tier alternating with those of the upper (see Plate 46, Figs. 4 and 5).

Combination of groynes and modassels.—Sometimes, when it is found necessary to protect a bank from immediate erosion and at the same time to divert the current from it, a series of groynes is put in with lengths of modassel in the intervals (see Plate 46, Fig. 6). With this arrangement the groynes are placed further apart than when there is no modassel work between them.

The cost of the groynes and modassels described ranges from Rs. 0-8-0 to Rs. 3 per foot, excluding carriage of materials, and in the Delta are well worth the expense if intelligently applied and properly cared for; but experience has shown that they are not well suited to a river of such depth and velocity as the Coleroon and they are now resorted to in that river only in cases of emergency. More drastic measures are needed to keep the Coleroon within bounds, and solid groynes of stone or concrete blocks and revetted spur banks are now the principal weapons of defence.

Protection against scour.—In old sepecifications for protective work it was always recommended that groynes and modassels should be protected from the undermining action of the obstructed current by means of "darbá rollers," but of late years this seems to have been seldom if ever done.

The darbá roller.—The "*darbá roller*" and "*darbá revetment*" are peculiar features of Tanjore river conservancy, though they have also been adopted in the Godavery and the Kistna.

The "darbá" roller is simply a large fascine of nánal, and is usually made up to a length of 24 feet with a diameter of 2½ feet or 3 feet Ropes of twisted nánal are laid parallel on the ground and a 2″ layer of nánal spread over them diagonally. A second layer is spread over this in the opposite direction and finally a 1½″ layer of earth or clay. The whole is then rolled up tightly and bound with the nánal ropes.

When used in combination with groynes and modassels one line of rollers is laid behind the legs of the horses and three or more lines, in the form of an apron, in front of the piles and, in the case of a groyne, round the nose. They are sunk flush with the river bed so as to cause no obstruction to the current and are finally picketted with 5 or 6 feet stakes (see Plate 45, Fig. 3).

Wattle wells.—Another device for preventing scour which was in vogue some years ago but which has not been made use of latterly is the "*wattle well*" This is simply a large gabion made up of small bamboos and the stems of "nir nochi" or other supple plant 3 to 4′ in diameter and 6′ high.

In the case of a groyne each leg of the outer horse is planted in one of these wells sunk flush with the river bed and filled in with sand, and 4 or 5 additional wells are sunk round the nose (see Plate 47, Fig. 2). In the case of a modassel each adjacent pair of legs is founded in a wattle well (see Plate 47, Fig. 3). These wells involve a good deal of time and labour and cost about Rs. 2 each and moreover cannot conveniently be used when there is much water, but when used they are undoubtedly a most effective protection against scour.

Darbá revetment.—A very efficient form of protection for earthen banks extensively used in Tanjore is "*darbá revetment*" (see Plate 47, Fig. 1).

This revetment is made up of nánal fascines about 6″ in diameter in the following manner :—

One or more large foot rollers, as already described, are laid along the toe of the bank to be revetted and staked down nearly flush with the bed. A layer of nánal, ¾ inch thick and 5 or 6 feet long, is then spread horizontally for a length of 20 to 30 feet over the foot roller, with half the length projecting beyond the required line of slope. A small fascine is then laid along the middle of this layer and the outer end of the latter is turned up, carried over the fascine and covered up with earth. Another flat layer is then spread over this and another fascine laid, and so on.

If the soil is suitable and the revetment is kept moist, the free ends of the nánal will take root and a very efficient protection will be formed. The cost of the facing runs to about Rs. 1-8-0 per square and of the foot roller to four annas per r. foot.

Conclusion.—This completes the list of conservancy methods which are believed to be peculiar to the Cauvery Delta or, what is practically the same thing, the Tanjore District, and which for the most part owe their origin to old native practice. Whether, as is quite possible, similar methods are in vogue in other provinces, the writer is not in a position to say.

PAPER No. 25.

Flood-drainage.

Engineering works constructed for purposes of flood-drainage consist mainly of open channels, sluices, weirs, and regulators, and in the present paper it is proposed to consider, as far as space will allow, the design of this class of works, with special reference to the calculations showing the discharge which they are capable of passing, under the conditions met with in practice. The text-books and standard works on the subject of Hydraulics give formulæ for the required discharges, under certain rather restricted conditions; and one of the first things that struck the writer when called on to design works of this nature, was that most of these conditions, under the circumstances met with in practice, were conspicuous by their absence. Two alternatives then presented themselves: the one, to proceed by precedent and rule-of-thumb; the other, to draw deductions from the standard formulæ, so as to adapt them to the more or less complicated conditions met with. Some of the results thus obtained have lately been presented in the form of a series of six lectures delivered by the writer at the Civil Engineering College, Sibpur, and these will shortly be published. In the limits of this paper it is not possible to give more than a mere sketch of the subject. First of all, it may be as well to indicate briefly the nature of the conditions met with in Bengal, where the writer's experience has been gained. The tracts in which large drainage schemes are required may be divided, broadly, into two classes, according to the nature of the outfall, *viz.*, (1) the inland tracts where the outfall is non-tidal, and (2) the tracts near the coast where outfall is tidal. The conditions in the inland tracts are comparatively simple. The areas to be drained consist, as a rule, of large swamps or jheels, flooded deeply in the rains, and draining out more or less completely in the dry season. There will, as a rule, be no particular difficulty in finding the direction of the natural drainage lines, and it will often suffice to simply clear out the natural outfall. In other cases a drainage cut, leading more directly to some main stream, will be preferable; and it may be necessary to provide sluices or regulators to exclude silt-bearing flood-water. In the tidal regions, matters are often far more complicated. The 24-Parganas District, south of Calcutta, is a good example. An area of some 200 square miles is entirely surrounded by an embankment, to exclude salt-water and storm-waves. The country is practically at a dead level throughout, that is, there is no general surface-slope of the land in any one direction. The natural exit-channels are silted up. The tides rise considerably above the level of the land; and the district is liable to occasional extraordinarily heavy falls of rain, the flood-water having to be led away through cuts made temporarily in the embankment, or through sluices which, owing to the range of the tides, can only discharge for a limited number of hours during the day. The soil is treacherous and full of springs, and care has to be taken that the sluices are not overworked, or they may be carried away. On the other hand, rigid economy in design has to be exercised owing to the poverty or want of foresight of the persons for whose benefit the works are constructed, and who, under the Bengal Embankment Act, have to pay the cost. It will easily be understood that great nicety of design is necessary to obtain the maximum available efficiency at a minimum cost.

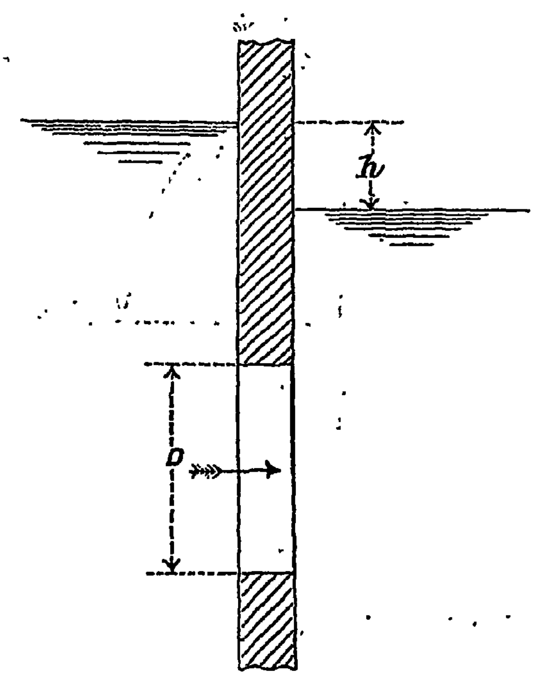
The theoretical aspect of the subject, which governs the calculations for the ventage of the sluices and the requisite sizes of the channels, can now be considered. The formulæ of the text-books with which we are mainly concerned are (*a*) those giving the discharges of sluices and weirs under constant heads, and (*b*) the discharges of open channels where the surface is parallel to the bed (longitudinally). The leading facts which give rise to complications are, mainly, that in these drainage schemes, more especially those in the tidal regions, (*a*) the heads over the sluices are seldom or never constant, and (*b*) the surfaces of the channels are often very far from parallel with the bed. Two other causes of complication may be mentioned, *viz.*, the existence of a velocity of approach, and the construction of a high breast-wall immediately in front of the vents of a sluice.

First, it is necessary to quote the formulæ referred to, giving the discharges of sluices and weirs under constant heads. There are five of them, of which two are simple and three are compound. The two simple ones are the typical formulæ of the sluice and the weir; the compound ones are made up of these two. All five fall into two general classes, *viz.*, the "sluice" class and the "weir" class. The formulæ are as follows:—

(i) *Completely submerged sluice (Fig. 1)*

$$q=c\sqrt{2g}\,bDh^{\frac{1}{2}} \qquad \text{(1)}$$

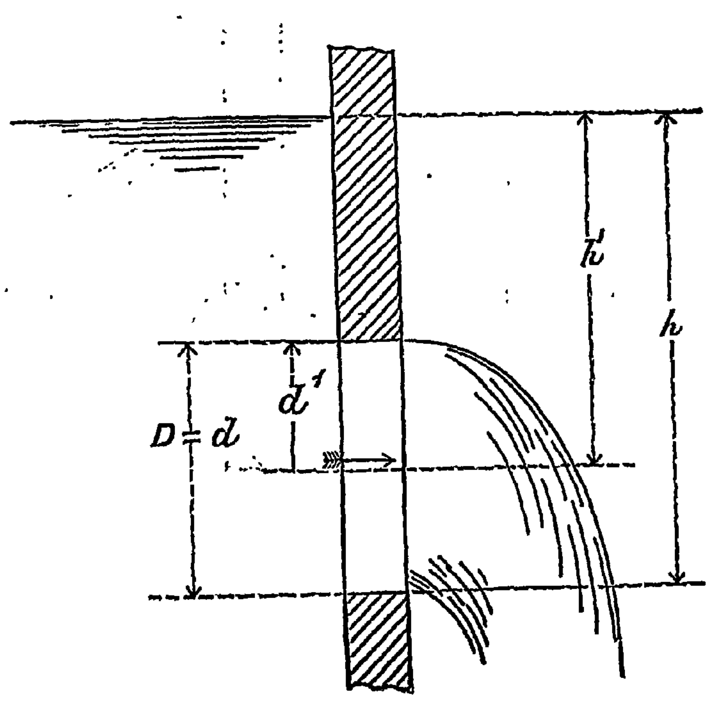
Fig: 1.



(ii) *Freely discharging sluice (Fig. 2)*

$$q=\frac{2}{3}\,c\sqrt{2g}\,b\left\{h^{\frac{3}{2}}-(h-D)^{\frac{3}{2}}\right\} \qquad \text{(2)}$$

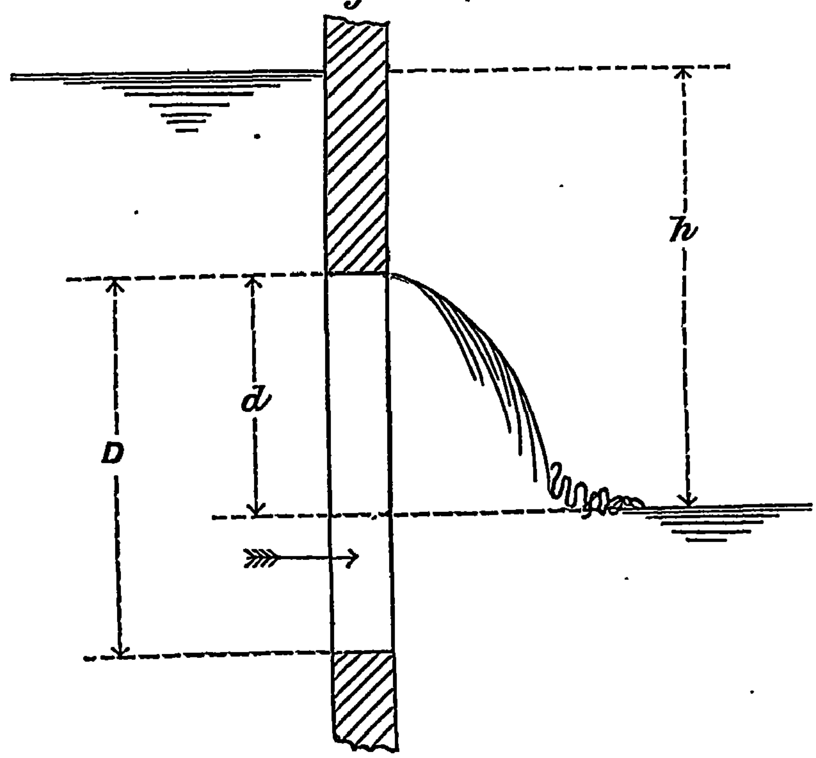
Fig: 2.



(iii) *Partially submerged sluice (Fig. 3)*

$$q = c\sqrt{2g}\, b\left[\frac{2}{3}\left\{ h^{\frac{3}{2}} - (h-d)^{\frac{3}{2}} \right\} + (D-d)h^{\frac{1}{2}}\right] \quad . \quad . \quad . \quad . \qquad (3)$$

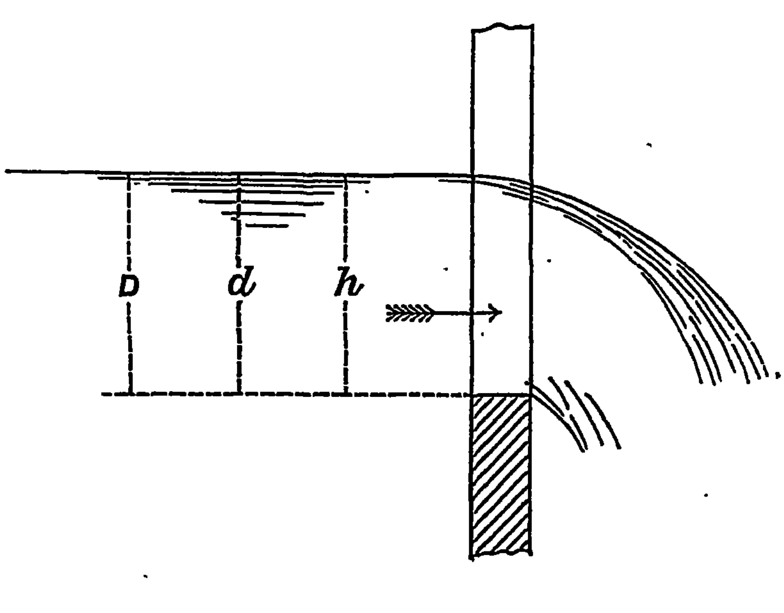
Fig. 3.



(iv) *Freely discharging weir (Fig. 4)*

$$q = \frac{2}{3}\, c\sqrt{2g}\; bh^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (4)$$

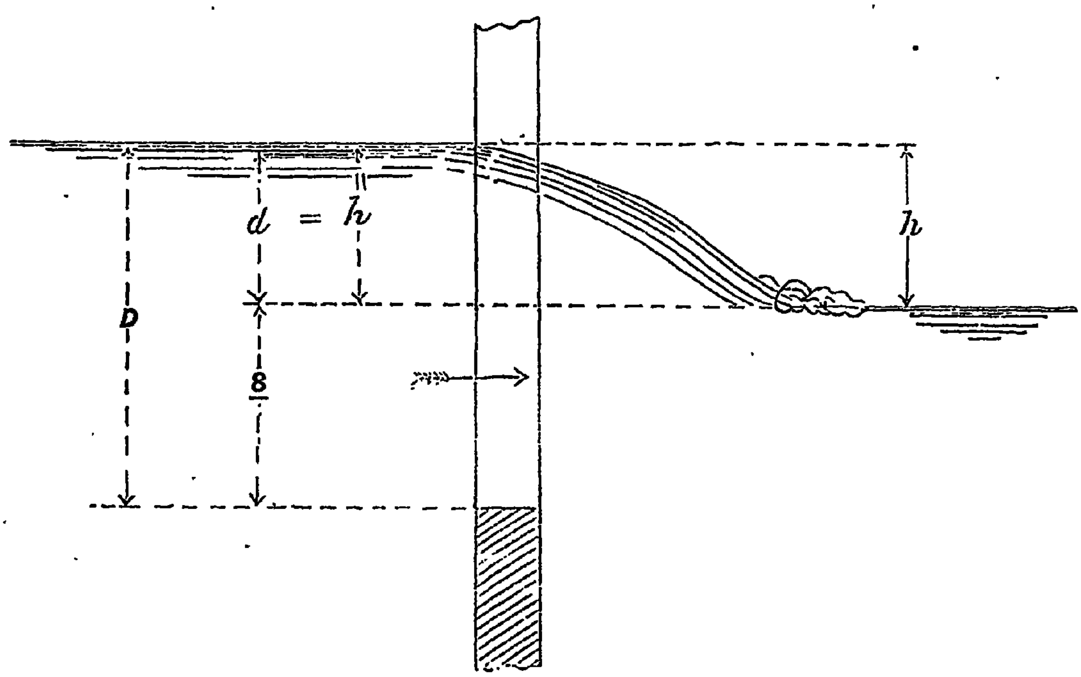
Fig. 4.



(v) *Drowned weir* (*Fig. 5*)

$$q = c\sqrt{2g}b\left\{\frac{2}{3}h^{\frac{3}{2}} + (D-d)\,h^{\frac{1}{2}}\right\} \qquad (5)$$

Fig. 5



It may here be pointed out that formula (iii) is the "general" form, from which all the others can be derived. It may be written in the form

$$q = F c\sqrt{2gb}\ Dh^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where F is a factor having the value

$$F = -\frac{d}{D}\,\frac{\frac{2}{3}\left(1-\frac{d}{h}\right)^{\frac{3}{2}} - \left(\frac{2}{3}-\frac{d}{h}\right)}{\frac{d}{h}}$$

and the ratios $\frac{d}{D}$ and $\frac{d}{h}$ may have any value from *0* to *1*. Thus, formula (i) is obtained by writing $\frac{d}{D} = 0$ and $\frac{d}{h} = 0$. When $\frac{d}{D} = 1$, while $\frac{d}{h}$ has any value between *0* and *1*, the result is formula (ii). The weir formula (iv) occurs when $\frac{d}{D}$ and $\frac{d}{h}$ are both equal to *1*; and the drowned weir when $\frac{d}{h} = 1$ while $\frac{d}{D}$ varies from *0* to *1*.

The arithmetical values of *F* are shown in Table I.

TABLE I.

The figures in the body of the Table are values of *F*, where

$$F = 1 - \frac{d}{D}\left\{\frac{\frac{2}{3}\left(1-\frac{d}{h}\right)^{\frac{3}{2}} - \left(\frac{2}{3}-\frac{d}{h}\right)}{\frac{d}{h}}\right\}$$

$d/D=$	0	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0
$d/h=$o	1·000	1·000	1·000	1·000	1·000	1·000	1·000	1·000	1·000	1·000	1·000
·1	1·000	·997	·995	·992	·990	·987	·985	·982	·980	·977	·975
·2	1·000	·995	·990	·984	·979	·974	·969	·964	·959	·953	·948
·3	1·000	·992	·984	·976	·968	·960	·952	·945	·937	·929	·921
·4	1·000	·989	·978	·968	·957	·946	·935	·924	·914	·903	·892
·5	1·000	·986	·972	·959	·949	·931	·917	·903	·890	·876	·862
·6	1·000	·983	·966	·949	·932	·915	·898	·881	·864	·847	·830
·7	1·000	·980	·959	·939	·918	·898	·878	·857	·837	·816	·796
·8	1·000	·976	·952	·928	·904	·879	·855	·831	·807	·783	·759
·9	1·000	·972	·943	·915	·887	·859	·830	·802	·774	·746	·717
1·0	1·000	·967	·933	·900	·867	·833	·800	·767	·773	·700	·667

It is, in many cases, desirable to obtain a general formula free from the complication of mathematical form exhibited by (6), and the following empirical formula will be found to give results varying by not more than 2 per cent. (and in most cases much less) from the accurate expressions given above. It is

$$q = c\sqrt{2g}\; b\mathrm{D} \left(h - \frac{5}{9}\frac{d^2}{\mathrm{D}}\right)^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

and it can also be written in the form

$$q = F\, c\sqrt{2g}\; b\mathrm{D}\, h^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

where F has the value

$$F = \left(1 - \frac{5}{9}\frac{d}{\mathrm{D}}\frac{d}{h}\right)^{\frac{1}{2}}$$

The arithmetical values of this factor are shown in Table II. A comparison of Tables I and II will show the relative accuracy of the formula (7).

Table II.

The figures in the body of the Table are values of F, where

$$F = \left(1 - \frac{5}{9}\frac{d}{\mathrm{D}}\frac{d}{h}\right)^{\frac{1}{2}}$$

$d/D=$	·0	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0
d/h	1·000	1·000	1·000	1·000	1·000	1·000	1·000	1·000	1·000	1·000	1·000
·1	1·000	·997	·994	·992	·989	·986	·983	·980	·978	·975	·973
·2	1·000	·994	·989	·983	·978	·972	·966	·960	·955	·949	·943
·3	1·000	·992	·983	·975	·966	·957	·949	·940	·938	·923	·907
·4	1·000	·989	·978	·966	·955	·943	·938	·919	·907	·894	·882
·5	1·000	·986	·972	·957	·943	·924	·907	·898	·882	·866	·850
·6	1·000	·983	·966	·949	·933	·907	·894	·876	·856	·837	·816
·7	1·000	·980	·960	·940	·919	·898	·876	·853	·830	·806	·782
·8	1·000	·978	·955	·938	·907	·882	·856	·830	·803	·775	·745
·9	1·000	·975	·949	·922	·894	·866	·837	·806	·775	·742	·707
1·0	1·000	·972	·943	·907	·882	·850	·816	·782	·745	·707	·667

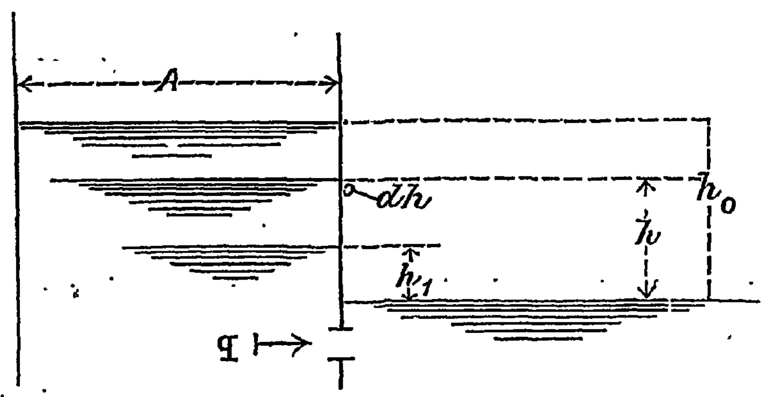
These results enable us to determine the "time of discharge" of sluices and weirs under varying heads. When water is being discharged through a sluice or weir from a basin of limited area into an outfall of unlimited extent, the surface of the basin will fall as the water runs out, while the surface of the outfall will be unaffected. Let A (Fig. 6) be the area of the basin; h the elevation of the basin surface above the outfall surface; q the rate of discharge; T the time taken by the surface in falling from h_0 to h_1; t the time taken in falling from h_0 to h. Then the quantity discharged by the sluice in time dt is qdt, and the quantity drawn off from the basin in the same time is Adh. We thus have the differential equation $qdt = -Adh$, which can be written

$$dt = -\mathrm{A}\frac{dh}{q} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

and the time t is obtained by integrating this equation, which involves the integration of the *reciprocal* of one of the five formulæ given above, whichever is suitable. The use of the approximate formula (7) will now be seen. It will be found convenient, in many cases, to determine the *true mean* rate of discharge throughout the time. Denoting this by q_m, and using q_o to denote the initial (maximum) rate of discharge, we have

$$q_m = \left(\frac{q_m}{q_o}\right) q_o$$

Fig. 6



and the determination of the factor $\frac{q_m}{q_o}$ will thus give the solution. In the present case q_m is the total quantity discharged, divided by the whole time, *i.e.*,

$$q_m = \frac{A\,(h_o - h_1)}{T}$$

where, as seen above, $T = -A\int_{h_o}^{h_1}\frac{dh}{q}$

Taking the five formulæ in detail, (1) is the well-known simple case of the time of discharge of a lock, and it will easily be seen that the results are—

$$T = \frac{2A}{c\sqrt{2gb}\,D}\left(h_o^{\frac{1}{2}} - h_1^{\frac{1}{2}}\right) \text{ and } \frac{q_m}{q_o} = \tfrac{1}{2}\left(1 + \frac{h_1^{\frac{1}{2}}}{h_o^{\frac{1}{2}}}\right) \qquad . \qquad (10)$$

The arithmetical values of this factor for various values of the ratio $\frac{h^1}{h_o}$ are shown in Table III.

TABLE III.

$\frac{h_1}{h_o} =$	·0	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0
$\frac{q_m}{q_o} =$	·50000	·65811	·72361	·77386	·81623	·85355	·88730	·91833	·94723	·97434	1·000

In the cases of formulæ (2) and (3) we must have recourse to the approximate formula (7), the result being

$$\frac{q_m}{q_o} = \tfrac{1}{2}\left\{1 + \frac{\left(h_1 - \frac{5}{9}\frac{d^2}{D}\right)^{\frac{1}{2}}}{\left(h_o - \frac{5}{9}\frac{d^2}{D}\right)^{\frac{1}{2}}}\right\} \qquad . \quad . \quad . \qquad (11)$$

Table III applies to this also if for the index we write $\dfrac{h_1-\frac{5}{9}\frac{d^2}{D}}{h_0-\frac{5}{9}\frac{d^2}{D}}$ instead of $\dfrac{h_1}{h_0}$

In the "weir" formulæ, the approximate expression is of no use, and (4) and (5) must be used. The result in the simple case is

$$\frac{q_m}{q_0}=\frac{1}{2}\left(1+\frac{h_1^{\frac{1}{2}}}{h_0^{\frac{1}{2}}}\right)\frac{h_1^{\frac{1}{2}}}{h_0^{\frac{1}{2}}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

and the arithmetical values are given in Table IV.

TABLE IV.

$\frac{h_1}{h_0}=$	0	·01	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0
$\frac{q_m}{q_0}=$	0	·055	·208	·323	·424	·516	·604	·687	·768	·847	·924	1·000

The drowned weir formula gives a less simple result, *viz.* :—

$$T=\frac{\sqrt{3}\,A}{c\sqrt{2g}\;b\sqrt{\delta}}\left[\tan^{-1}\left\{\left(\frac{2h_0}{3\delta}\right)^{\frac{1}{2}}\right\}-\tan^{-1}\left\{\left(\frac{2h_1}{3\delta}\right)^{\frac{1}{2}}\right\}\right] \quad . \quad . \quad . \quad . \quad (13)$$

The arithmetical results of the factor in brackets are given in Table V.

TABLE V.

The figures in the body of the Table are values of the factor.

$$\tan^{-1}\left\{\left(\frac{2h_0}{3\delta}\right)^{\frac{1}{2}}\right\}-\tan^{-1}\left\{\left(\frac{2h_1}{3\delta}\right)^{\frac{1}{2}}\right\}$$

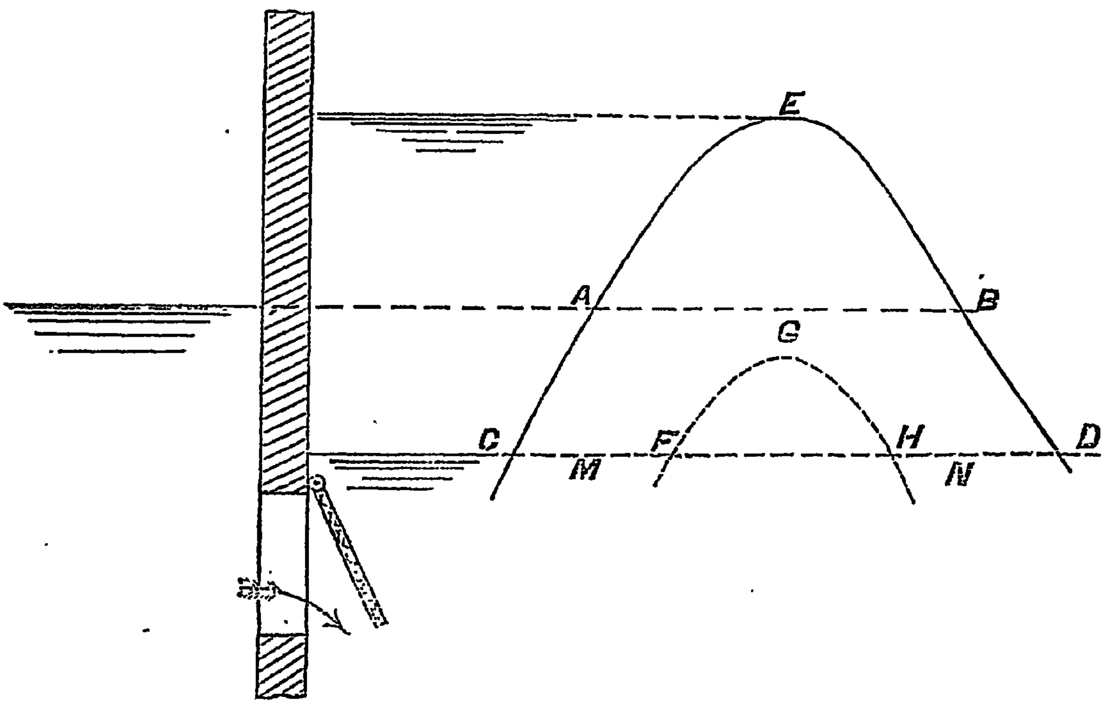
$\frac{\delta}{\delta+h_0}=$	...	0	·01	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0
$\frac{2h_0}{3\delta}=$	...	∞	66	6	2·6667	1·5555	1·0	·66667	·44444	·28571	·16667	·07407	0
$\frac{\delta}{\delta+h}$	$\frac{2h_1}{3\delta}$												
0	∞	0											
·01	66	·12247	0										
·1	6	·38747	·26504	0									
·2	2·6667	·54949	·42702	·16202	0								
·3	1·5555	·67574	·55327	·29827	·12625	0							
·4	1·0	·78540	·66293	·39793	·23591	·10966	0						
·5	·66667	·88605	·76358	·49858	·33656	·21031	·10065	0					
·6	·44444	·98262	·86015	·59515	·43313	·30688	·19722	·09657	0				
·7	·28571	1·07978	·95731	·69231	·53029	·40404	·29438	·19373	·09716	0			
·8	·16667	1·18305	1·06058	·79558	·63356	·50731	·39765	·29700	·20043	·10327	0		
·9	·07407	1·30493	1·18246	·91746	·75543	·62919	·51953	·41888	·32231	·22515	·12188	0	
1·0	0	1·57080	1·44833	1·18333	1·02131	·90506	·78540	·68475	·58818	·49102	·36775	·26587	0

We now come to a set of conditions which is of very frequent occurrence in Bengal, and is of importance, because the results enter into almost every case of drainage into tidal outfalls. I allude to the cases where the surface of the head-water falls at so slow a rate, compared with the rise or fall of the tail-water, that it may be taken as practically at a constant level, while the outfall rises and falls with the tide. In schemes for the drainage of large agricultural areas in Bengal, the rate at which it is usually considered necessary to lower the surface of the drained area is three-quarters of an inch in 24 hours whereas the rate of rise of the tide may be as much as three feet in one hour, *i.e.*, more than a thousand times as fast.

The nature of the tide-curve is an important element in the solutions. The tidal *data* form a part of the preliminary observations necessary to the design of these drainage schemes, and the tidal curve is constructed by setting off the times as abscissæ and the heights of the gauge as ordinates, to any convenient scale. For the tidal rivers of Lower Bengal the result is of the type shown in Figure 18. The front of the curve, representing the rise of the tide, is nearly parabolic, and so is the portion corresponding with the first part of the ebb. Then the curvature changes, and the remainder of the ebb corresponds fairly nearly with an inverted parabola, but with a rather flatter curvature. The duration of the ebb is, roughly, about double that of the flood.

All drainage-sluices in these waters are fitted with flap-shutters outside the vents, so as to allow drainage-water to pass out, but to close automatically as soon as the tide rises above the level of the interior water. Thus the sluice will only discharge while the tide is below the level of the water inside the sluice. The diagram in Figure 7.

Fig 7.

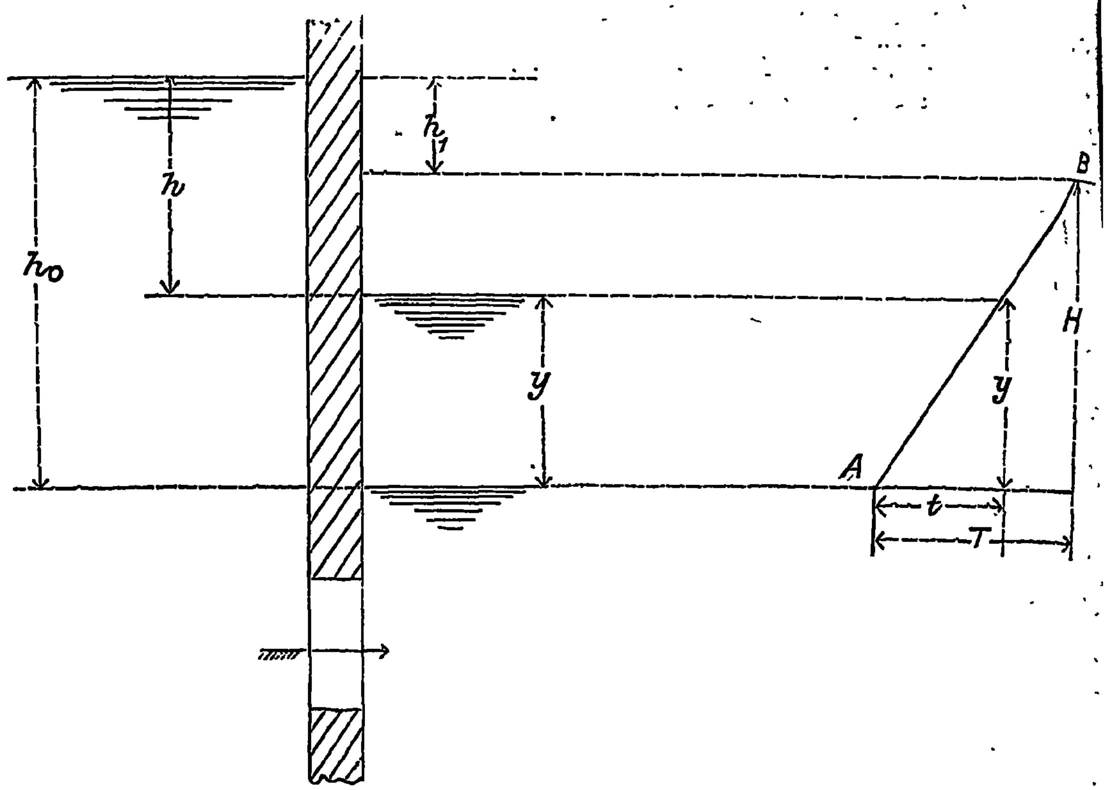


represents a sluice, inside which the water is standing at the level AB. $CAEBD$ is the tide-curve, E being the level of high-water. Suppose the tide is rising, and the sluice is opened when the tide-level stands at CD. Then the discharge will continue, at a gradually decreasing rate, until the tide has risen as high as AB, when the flap-shutter will close and discharge will cease. The sluice will remain closed until the tide falls again to AB, when the shutter will again open, and allow the discharge to pass, at a gradually increasing rate, until the tide again falls to CD. Now the *time* during which the sluice remains open is represented on the tide-curve by the lengths $(CM+ND)$. Using q_o to denote the discharge due to the "head" AM, the *mean* discharge throughout the time will be, as noted above

$$q_m = \left(\frac{q_m}{q_o}\right) q_o$$

and we have to determine the value of the factor $\frac{q_m}{q_0}$. It should be noted that, in the curve $CAEBD$, the portions CA and BD are very nearly straight lines, indicating that the tide rises and falls at a very nearly uniform rate. When, however, high water is at or below the level of the inside water, the tide-curve will be somewhat as shown by the dotted line FGH, and its parabolic form will have to be taken into account. Coming to details, the simplest case (Figure 8)

Fig. 8.



occurs when the "head" over a completely-submerged sluice is gradually extinguished, at a uniform rate, by the rising tide. The rate of rise being uniform, the tide-curve is a straight line, with the equation

$$y = \frac{H}{T}t \qquad (14)$$

The discharge after time t is

$$q = c\sqrt{2g}\, b\, D\, (h_0 - y)^{\frac{1}{2}} \qquad (15)$$

and the mean discharge in time T is

$$q_m = \frac{1}{T}\int_0^T q\,dt \qquad (16)$$

Hence $q_m = \frac{1}{T}\, c\sqrt{2g}\, bD \int_0^T \left(h_0 - \frac{H}{T}t\right)^{\frac{1}{2}} dt.$

Integrating, and dividing the result by q_0, the value of which is, of course, $c\sqrt{2g}\, bDh_0^{\frac{1}{2}}$, the result is

$$\frac{q_m}{q_0} = \frac{2}{3}\,\frac{1-\left(1-\frac{H}{h_0}\right)^{\frac{3}{2}}}{\frac{H}{h_0}} \qquad (17)$$

The arithmetical values of $\frac{q_m}{q_o}$ for various values of $\frac{H}{h}$ are shown in Table VI:—

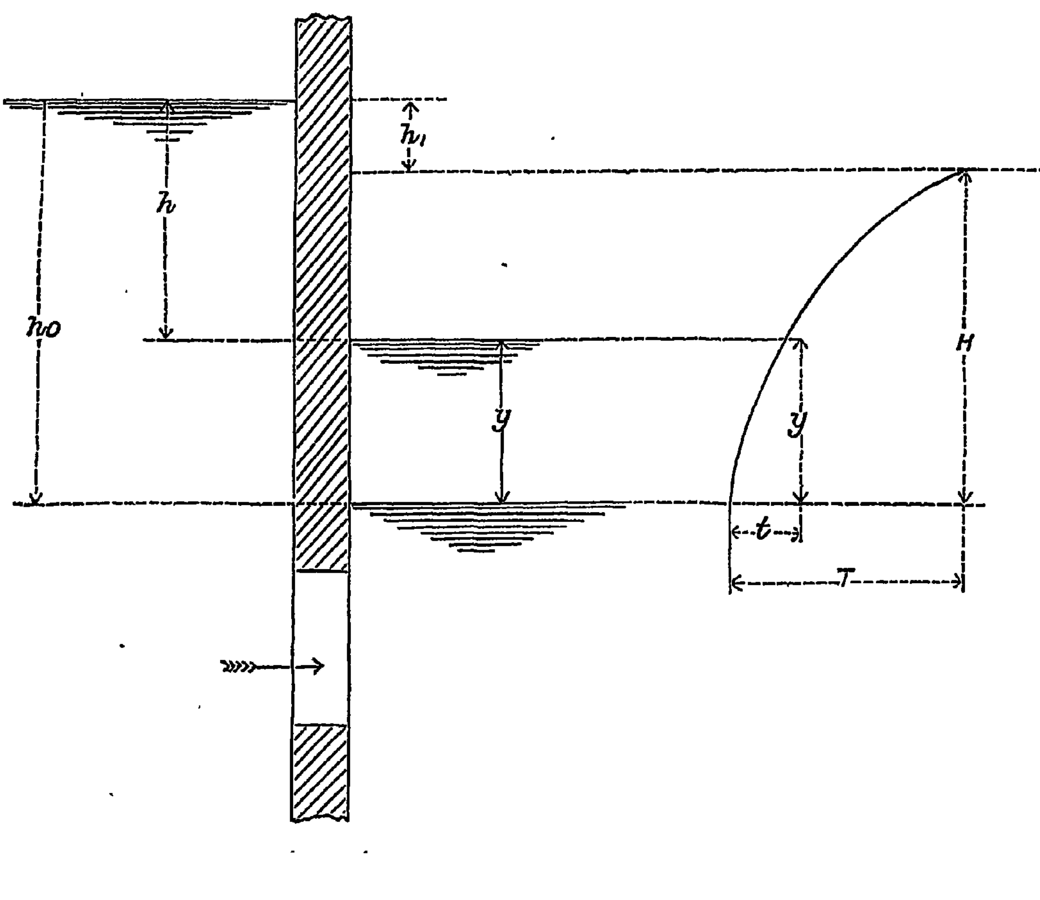
TABLE VI.

$\frac{H}{h_o}=$	0	·02	·05	·10	·2	·3	·4	·5	·6	·7	·8	·9	·95	1·0
$\frac{q_m}{q_o}=$	1·00000	·99400	·98760	·97460	·94828	·92075	·89207	·86193	·83002	·79588	·75880	·71762	·69392	·66067

It may be noted that when $\frac{H}{h_o}$ is 1, then the value of the factor is $\frac{2}{3}$; that is, when the head is completely extinguished by a uniformly rising tide, the mean discharge is two-thirds of the maximum.

When high-water level is at or below the level of the head-water (Fig. 9),

Fig. 9.



the parabolic shape of the tide-curve has to be allowed for, and instead of equation (14) the value of y has to be obtained from the parabolic equation

$$\frac{H-y}{H}=\left(\frac{T-t}{T}\right)^2 \qquad (18)$$

Combining this with equations (15) and (16) and integrating, the result is

$$\frac{q_m}{q_o}=\frac{1}{3}\left\{1+\frac{1}{2}\frac{\frac{h_o}{H}-1}{\sqrt{\frac{h_o}{H}}}\log_e\left(\frac{\sqrt{\frac{h_o}{H}}+1}{\sqrt{\frac{h_o}{H}}-1}\right)\right\} \qquad (19)$$

The arithmetical values of this factor are shown in Table VII.

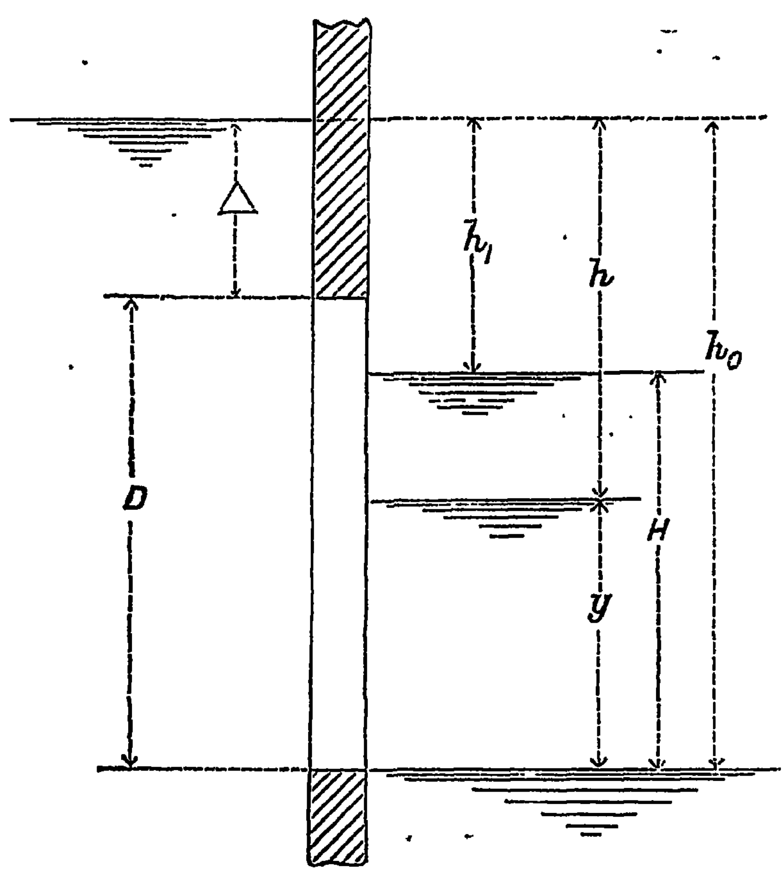
TABLE VII.

H/h_0 = .	0	·02	·05	·10	·2	·3	·4	·5	·6	·7	·8	·9	·95	1·0
h_0/H = .	∞	50	20	10	5	3·33333	2·50000	2	1·66667	1·4286	1·25	1·1111	1·0526	1·0
q_m/q_0 = .	1·00000	·99330	·98316	·96593	·93040	·89299	85316	·81168	·76639	·71639	·66143	·59582	·55584	·50000

It will be seen that the effect of allowing for the parabolic shape of the tide-curve is to reduce the factor, in the case of complete extinction of the head, from $\frac{2}{3}$ to $\frac{1}{2}$.

The remaining sluice cases and the weir cases can be solved from the general case of a partially submerged sluice. Fig. 10

Fig. 10.



represents a sluice where the initial outfall-level coincides with the floor of the vent. The initial head is h_0, the final head h_1, and the head-water stands at a height Δ above the top of the vent. Taking first the case where the tide is rising uniformly, the value of y is the same as in equation (14). The discharge after time t is

$$q = c\sqrt{2g}\; b\left\{(h_0 - h)\,h^{\frac{1}{2}} + \frac{2}{3}\left(h^{\frac{3}{2}} - \Delta^{\frac{3}{2}}\right)\right\} \qquad (20)$$

And since $h = h_o - y$, it follows that

$$\frac{dh}{dt} = -\frac{dy}{dt} = -\frac{H}{T}$$

Now equation (16) must be put in the form

$$q_m = \frac{1}{T}\int_0^T q\frac{dt}{dh}\,dh = -\frac{1}{T}\,\frac{T}{H}\int_{h_o}^{h_1} q\,dh \qquad . \quad . \quad . \quad (21)$$

The result is of course obtained by substituting the value of q from (20) and (21) and integrating. The final result is

$$\frac{q_m}{q_o} = \frac{\left\{1-\left(\frac{h_1}{h_0}\right)^{\frac{3}{2}}\right\} - \frac{1}{5}\left\{1-\left(\frac{h_1}{h_0}\right)^{\frac{5}{2}}\right\} - \left(\frac{\Delta}{h_o}\right)^{\frac{3}{2}}\left(1-\frac{h_1}{h_o}\right)}{\left(1-\frac{h_1}{h_o}\right)\left\{1-\left(\frac{\Delta}{h_o}\right)^{\frac{3}{2}}\right\}} \qquad . \; (22)$$

The arithmetical values are given in Table VIII :—

TABLE VIII.

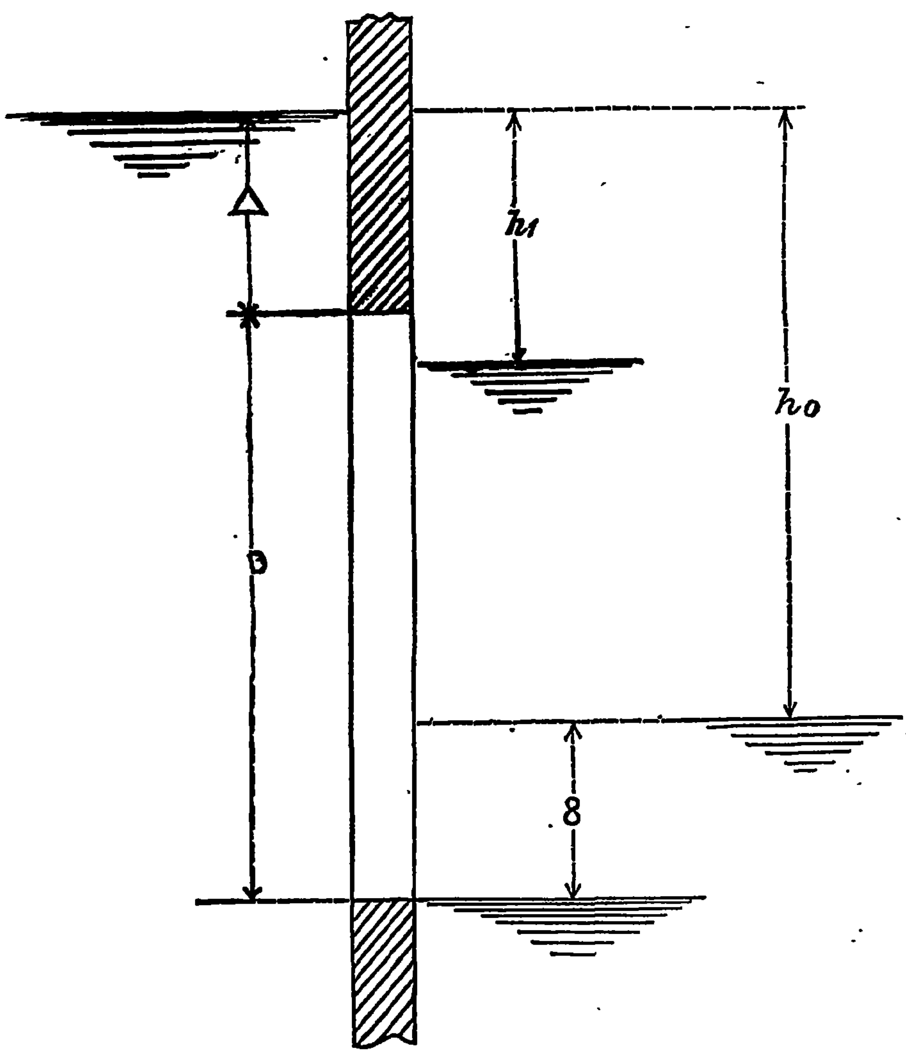
$h_1/h_o =$	0	·1	·2	·3	·4	·5	·6	·7	·8	·9	1 0
$H/h_o =$	0	·9	·8	·7	·6	·5	·4	·3	·2	·1	0
$\Delta/h_o = 0$	·80000	·85446	·89368	·92220	·94543	·96360	·97752	·98778	·99480	·99874	1·000
·1		·84960	·88016	·92964	·94365	·96211	·97679	·98783	·99458	·99867	1·000
			·89225	·91456	·91007	·96002	·97530	·98753	·99431	·99861	1·000
·3				·90690	·93470	·95644	·97331	·98533	·99376	·99846	1·000
·4					·92696	·95123	·96991	·98358	·993 05	·99828	1·000
·5						·94867	·96523	·98102	·99197	·99807	1·000
·6							·95799	·97708	·99028	·99749	1·000
·7								·97098	·98744	·99678	1·000
·8									·98170	·99552	1·000
·9										·99118	1·000
1·0											1·000

The case of weir-discharge occurs when $\Delta = 0$, so that the values applicable will be found in the top line. When in addition the degree of submergence is complete, *i.e.*, when the head over a weir is completely extinguished by the rising tide, the mean discharge is seen to be $\frac{4}{5}$ths of the maximum.

We now have to deal with the case (Fig. 11) where the initial outfall level is above the floor of the sluice. Using δ to denote the initial depth of submergence, and h_0 the initial head, which is *not* measured down to the floor-level, the sluice may be treated as made up to two portions, of which the top portion, of depth $(D-\delta)$ acts as a sluice discharging freely to start with, and gradually submerged, just as in the case last considered. The lower portion, of depth δ, acts as a completely submerged sluice during the whole time. Now, writing q_{01} for the initial discharge of the upper portion, the mean discharge of this portion throughout the time may be represented by q_{m1} where $q_{m1} = f_1 q_{01}$ f_1 being the factor obtained from Table VIII using the index-values $\frac{h_1}{h_o}$ and $\frac{\Delta}{h_o}$. Similarly the mean discharge of the lower portion may be represented by q_m where $q_{m2} = f_2 q_{o2}$ and f_2 is the factor obtained from Table VI, using the

index-value $\frac{h_0 - h_1}{h_0}$, which is the same as $\frac{H}{h_0}$; and q_{02} is the initial discharge of

Fig. 11.



the lower portion. Thus the mean discharge of the whole sluice is

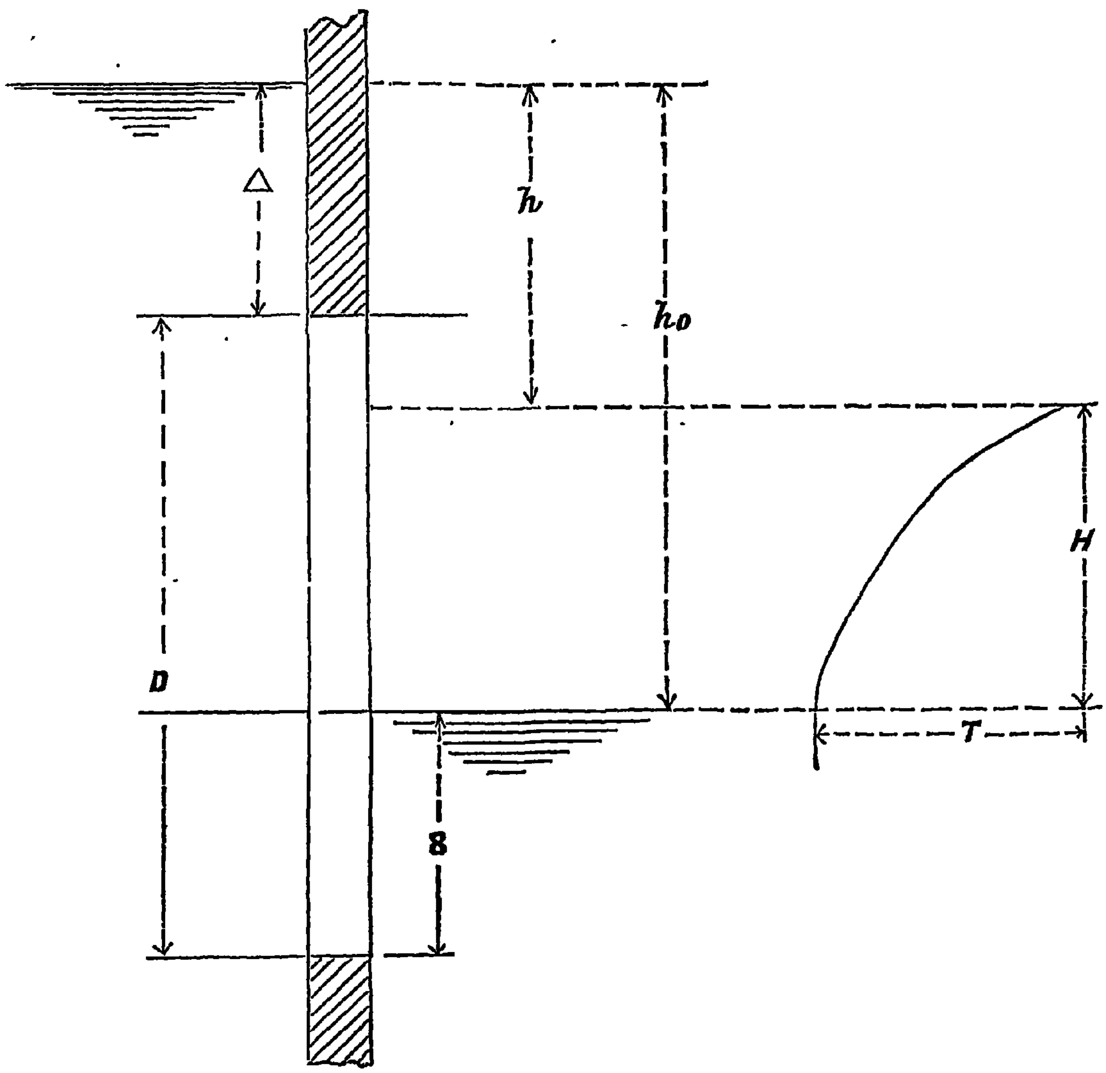
$$q_m = q_{m1} + q_{m2} = f_1 q_{01} + f_2 q_{02}$$

and since the initial discharge is $q_0 = q_{01} + q_{02}$, the final factor is

$$\frac{q_m}{q_0} = \frac{f_1 q_{01} + f_2 q_{02}}{q_{01} + q_{02}} \quad . \quad . \quad . \quad . \quad (23)$$

When the level of high water is at or below the top of the vent, the parabolic shape of the tide-curve has to be allowed for (Fig. 12).

Fig. 12.



Equation (21) may be expressed in the form

$$q_m = \frac{1}{T} \int_{h_0}^{h_1} \frac{g}{\frac{dh}{dt}} \, dh \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

where the value of $\frac{dh}{dt}$ is $-\frac{dy}{dt}$, obtained from the parabolic equation (18); and that of q is obtained from the discharge equation (20). The final result is as follows:—

$$\frac{q_m}{q_0} = \frac{1}{1-\left(\frac{\Delta}{h_0}\right)^{\frac{3}{2}}} \left[\cdot 62500 - \cdot 18750 \left(1 - \frac{H}{h_0}\right) - \left(\frac{\Delta}{h_0}\right)^{\frac{3}{2}} + \frac{\cdot 09375 \left(3 + \frac{H}{h}\right) \left(1 - \frac{H}{h_0}\right)}{\left(\frac{H}{h_0}\right)^{\frac{1}{2}}} \log e \left\{ \frac{1 + \left(\frac{H}{h_0}\right)^{\frac{1}{2}}}{1 - \left(\frac{H}{h_0}\right)^{\frac{1}{2}}} \right\} \right] \quad . \quad (25)$$

or, the quantity $(h_o - h_1)$ may be substituted for H, and the result expressed in terms of $\frac{h_1}{h_o}$ instead of $\frac{H}{h_o}$. The arithmetical results are shown in Table X.

TABLE IX.

$h_1/h_o =$	0	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0
$H/h_o =$	1·0	·9	·8	·7	·6	·5	·4	·3	·2	·1	0
$\Delta/h_o = 0$	·62500	·74611	·81719	·86716	·90962	·94024	·96338	·98017	·99150	·99791	1·00000
·1		·73813	·81152	·86312	·90565	·93827	·96219	·97951	·99122	·99784	1·00000
·2			·79956	·85444	·90074	·93457	·95973	·97823	·99067	·99770	1·00000
·3				·84140	·89165	·92830	·95616	·97627	·98983	·99750	1·00000
·4					·87903	·92001	·95093	·97346	·98862	·99722	1·00000
·5						·90757	·94337	·96933	·98687	·99678	1·00000
·6							·93158	·96294	·98412	·99610	1·00000
								·95214	·97947	·99495	1·00000
·8									·97011	·99264	1·00000
·9										·98501	1·00000
1·0											1·0000

As before, the top line applies to weir-discharges. It will be seen that in the case of a weir just completely submerged at high water, the effect of the parabolic curvature is to reduce the mean discharge from $\frac{4}{5}$ ths to $\frac{5}{8}$ ths of the maximum.

When the sluice is initially submerged to a depth δ over the floor the solution is obtained in the same way as before, *i.e.*

$$\frac{q_m}{q_o} = \frac{f_1 q_{o1} + f_2 q_{o2}}{q_{o1} + q_{o2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (26)$$

where f_1 is the factor obtained from Table IX, using the index values $\frac{h_1}{h_o}\left(\text{or } \frac{H}{h_o}\right)$ and $\frac{\Delta}{h_o}$; f_2 is the factor obtained from Table VII; q_{o1}, the discharge of the upper portion, is equal to $\frac{2}{3}\, c\sqrt{2gb}\, \left(h_o^{\frac{3}{2}} - \Delta^{\frac{3}{2}}\right)$; and q_{o2}, the discharge of the lower portion, is equal to $c\sqrt{2\,g}\; b\; \delta\; h_o^{\frac{1}{2}}$.

There are two more classes of varying head, which need not be dealt with here, *viz.*, (1) when the discharge occurs between two basins, both of restricted area, so that the lower basin is filled as the upper one empties; and (2) when the discharge occurs from a small basin into a tide-way, so that the outfall surface varies independently of the fall of the head-water. This can only occur in the case of small basins, such as locks or dock-basins, as if the basin is large, the fall of the head-water occurs at too slow a rate to be considerable.

The existence of a "velocity of approach" is sometimes a troublesome factor in calculations, and any simplification of the usual method of dealing with it is useful. The usual way is to add to the "head" in the discharge formula a quantity termed the "head due to velocity of approach," an undetermined quantity, necessitating the use of trial methods to work out the value of the discharge. This unknown quantity can, however, be eliminated, as follows. The velocity of approach we are here concerned with occurs when the water is led up to the sluice along a channel of restricted area. Calling the cross sectional area of the channel Ω, the velocity of approach u, and the ventage of the sluice a, the value of u is clearly $u = \frac{q}{\Omega}$ and the discharge equation (in the case of a submerged sluice) is $q = c\sqrt{2g}\; a\, \left(h + \frac{u^2}{2g}\right)^{\frac{1}{2}}$. Now,

substituting for u the above value, an equation is obtained which can be reduced to the form

$$q=C_s\, ca\sqrt{2g}\; h^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

where C_s has the value

$$C_s=\frac{1}{\left(1-\frac{c^2\,a^2}{\Omega^2}\right)^{\frac{1}{2}}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (28)$$

The arithmetical values of C_s for various values of the ratio $\frac{ca}{\Omega}$ are shown in Table X.

The case of weir-discharge is rather more involved. The fundamental equation is

$$q=\frac{2}{3}cb\sqrt{2g} \qquad \left\{\left(h+\frac{u^2}{2g}\right)^{\frac{3}{2}}-\left(\frac{u^2}{2g}\right)^{\frac{3}{2}}\right\}$$

Writing $\mathfrak{b}$ for $\frac{u^2}{2g}$, this equation can be written as follows :—

$$q=C_w\,\frac{2}{3}cb\sqrt{2gh^{\frac{3}{2}}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (29)$$

where the value of C_w is

$$C_w = \left\{\left(1+\frac{\mathfrak{b}}{h}\right)^{\frac{3}{2}}-\left(\frac{\mathfrak{b}}{h}\right)^{\frac{3}{2}}\right\} \quad . \quad . \quad . \quad . \quad (30)$$

Now, the value of q is

$$q = \Omega\, u = \Omega\,(2g\;\mathfrak{b})^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

Equating (29) and (31), and using a to denote the area of waterway bh, the following relation is obtained :—

$$\frac{ca}{\Omega}=\frac{3}{2C_w}\left(\frac{\mathfrak{b}}{h}\right)^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

The corresponding values of $\frac{ca}{\Omega}$ and C_w are shown in Table X, which is calculated as follows :— Select any convenient values of $\frac{\mathfrak{b}}{h}$, ranging from 0 upwards, and for each value calculate first the value of C_w from (30) and next the value of $\frac{ca}{\Omega}$ from (32). Tabulate all the values thus obtained. The val ues of $\frac{\mathfrak{b}}{h}$ are not required, except as stepping-stones to the values of the other two quantities. Any required intermediate values of $\frac{ca}{\Omega}$ and C_w can be obtained by proportional parts, or by plotting the calculated values, drawing curves through them, and scaling off the required values. The accuracy of the results can be tested by trying whether the values in Table X satisfy both equations (30) and (32). It will be seen that the values of C_w correspond fairly nearly with those of C_s. The factor C_s may be used for all the " sluice " cases, *i.e.*, when the top of the vent is below the surface of the head-water.

TABLE X.

$\frac{ca}{\Omega}=$	0	·05	·10	·15	·20	·25	·30	·35	·40	·45	·50	·55	·60	·65	·70	·75	·80	·85	·90	·95	1·00
$C_s=$	1·000	1·0013	1·005	1·0116	1·021	1·033	1·049	1·069	1·091	1·120	1·155	1·198	1·250	1·310	1·400	1·512	1·667	1·898	2·204	3·203	∞
$C_w=$	1·000	1·0015	1·006	1·0110	1·028	1·040	1·050	1·081	1·109	1·141	1·180	1·229	1·290	1·361	1·453	1·572	1·836	2·012	2·440	3·83	∞

A complicated case of discharge occurs when a breast-wall is constructed in front of the vents of a sluice. The object of doing this is to limit the high velocity which would otherwise occur through the sluice at low water. Suppose the floor of a sluice is at zero, and that there is a breast-wall 8 feet high on the upper side of the vents, the flood-level on the country standing at + 12·00. Then the velocity can never exceed that due to a weir-discharge under a head of 4 feet; whereas, in the absence of the breast-wall, it would increase, at low tide, to perhaps something like 20 feet per second. The danger from excessive velocity arises not only from the vibration caused, but from the scouring out of the outfall below the sluice, which may extend backwards and undermine the foundations, or into which the sluice may be forced bodily by the pressure. The complete determination of this case is too long to reproduce here, but in practice an approximation may be employed, consisting in making the calculation as if the vents did not exist, and the discharge occurred simply over the breast-wall, as over a weir. In designing the sluice the waterway through the vents should be made at least as large as the maximum waterway over the weir. The results can be checked, and corrected, if necessary, by the more exact method. To proceed with the design. We know from the tidal curve the number of hours, during the rise and fall of one tide, that the tidal level is below the crest of the weir, *i.e.*, that the weir-discharges "free." Denote this by t_o. Suppose that, in this case, the tide rises well above the level of the head-water so that the rate of rise of the tide is approximately uniform. Let t_1 denote the number of hours occupied by the tide in rising from the crest-level of the weir to the level of the head-water (*i.e*, time taken in extinguishing the head over the weir) and in falling again between those levels. The time during which the tidal level is above the head-water and discharge is completely blocked, may be denoted by t_2. Thus, for a tide of 13 hours, we have $t_o + t_1 + t_2 = 13$. We have seen above that the true mean discharge of the weir during the time t_1 is equal to $\frac{4}{5}q_o$ where q_o is the maximum discharge. Calling q_n the mean discharge throughout the whole tide, we have—

$$q_n\ (t_o+t_1+t_2)=q_o\ t_o+\tfrac{4}{5}q_o\ t_1+0$$

$$\text{That is, } q_n=\frac{t_o+\frac{4}{5}t_1}{13} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (33)$$

Suppose now it is desired to design a sluice of sufficient capacity to lower the flood-level over an area of M square miles at the rate of r inches in 24 hours. It is a matter of arithmetic that a flow-off of 1 inch in 24 hours from 1 square mile is equal to a continuous discharge of 26·89 *c.f.s.* during that time, so that $q_n=26{\cdot}89\ Mr$. Substituting this value in equation (33), the value of q_o is—

$$q_o=\frac{350\ Mr}{t_o+\frac{4}{5}t_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (34)$$

The breast-wall must be of sufficient width to pass this discharge, when discharging "free." The discharge depends also, of course, on the depth of water passing over. The depth which may safely be allowed to pass over is a matter for the judgment of the designer. In sluices built on the treacherous soil which is usually met with in Lower Bengal, 4 feet is probably as great a depth as is advisable. Calling the depth D, the maximum discharge is, with the usual coefficient,

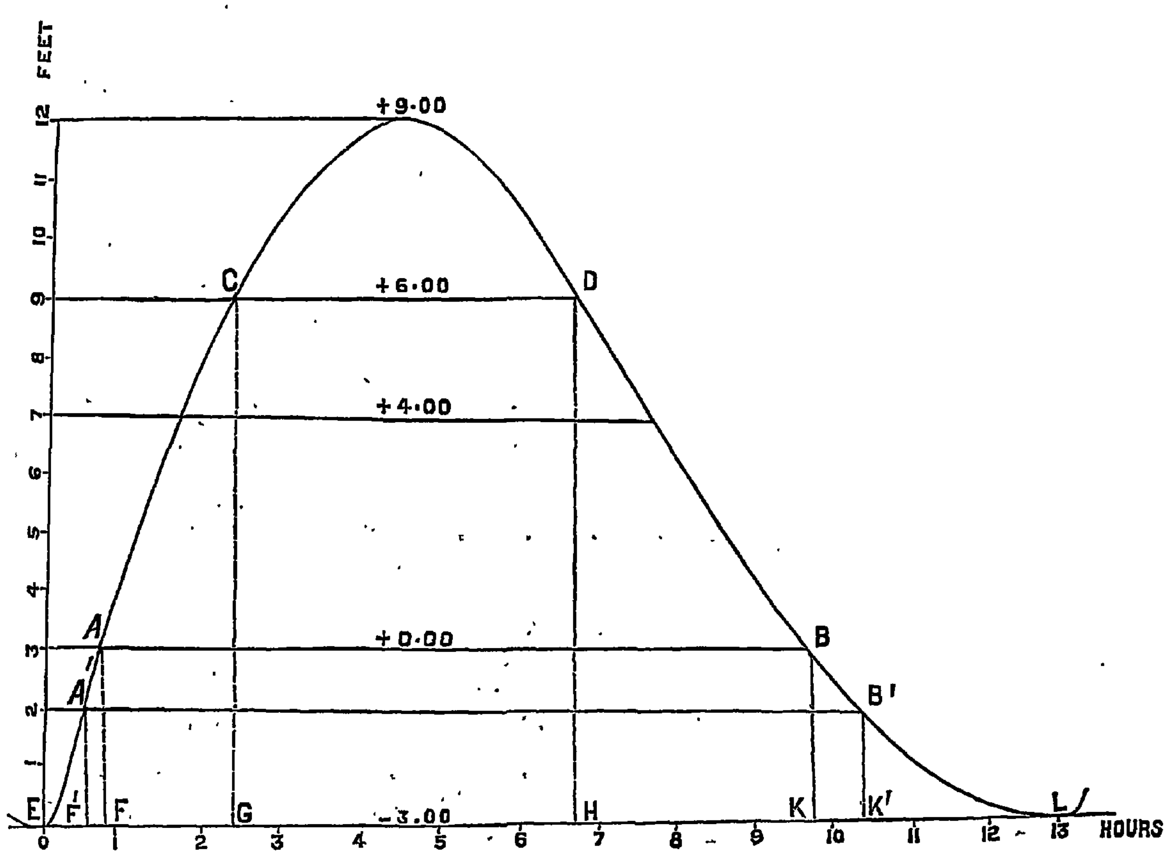
$$q_o=\frac{10}{3}\,b_w\,D^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

combining this with (34), the width is determined thus —

$$b_w=\frac{105\ Mr}{\left(t_o+\frac{4}{5}t_1\right)D^{\frac{3}{2}}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

The foregoing results may be exemplified as follows. Suppose it is desired to drain a swamp whose level is 6 feet above M. S. L. by means of a sluice discharging into a tide-way where the tidal curve has a range from 9 feet above to 3 feet below M. S. L. The water has to be led from the swamp to the sluice by an open channel 4 miles long. The area to be drained is 30 square miles, and the level has to be lowered at the rate of $\frac{3}{4}$ of an inch in 24 hours. The levels which are known are those of the swamp and the tide-way; and the dimensions of the channel have to be decided on, as well as those of the sluice. t is clear that, the steeper the slope of the channel, the smaller will be its necessary width and depth, and *vice versâ*. On the other hand, a steep slope in the channel will involve a lower level at its tail, a less efficient "head" over the sluice, and greater ventage-area. Also, the sluice will be blocked for a longer time, necessitating a higher discharge while it is working, and greater capacity of both sluice and channel. The most economical slope can only be determined by trial, but suppose we decide in this case to adopt a slope of 6 inches per mile. The dimensions of the channel may be considered presently, but so far as the sluice is concerned the drop of level in the 4 miles will be 2 feet, giving a level inside the sluice of + 4·00. Suppose, again, it is decided to allow 4 feet depth of water to pass over the breast-wall. Then this fixes the level of the weir-crest at 0·00. Fig. (13)

Fig. 13.



represents the tidal curve. The discharge will be "free" while the tide is below the level 0·00, that is during the time represented by $(EF+KL)$. This corresponds with the t_0 of equation (35). It has to be noted that, as the tide rises and backs up the discharge, the water at the lower end of the channel will rise at the same time. For the purposes of this calculation we may assume that the discharge will be reduced at a uniform rate until the tide rises to the level of the swamp, *viz.*, to + 6·00. The time t_1 is thus

$(FG+HK)$. The time during which the sluice is blocked is GH. We can now find the value of b_w from equation (36), since we have all the *data*, *viz.*:

$$M=30\ ;\ r=\frac{3}{4}\ ;\ t_o=4\ ;\ t_1=4{\cdot}6\ ;\ D=4$$

$$b_w=\frac{105\times30\times\frac{3}{4}}{(4+\frac{4}{5}\times4{\cdot}6)\times4\frac{1}{2}}=38{\cdot}45\text{, or say 39 feet.}$$

That is, the weir must be at least 39 feet wide; and, with a depth of 4 feet of water passing over it, the waterway will be ($4\times39=$) 156 square feet. The sluice may therefore be given four vents, each 5 feet wide by 8 feet high. The maximum and mean discharges may also be noted, *viz.*:—

$$q_m=26{\cdot}89\ ;\ Mr=605\ c.f.s.\text{, and (from 34)}$$

$$q_o=\frac{350\times30\times\frac{3}{4}}{4+\frac{4}{5}\times4{\cdot}6}=1{,}025\ c.f.s.$$

As regards the design of the channel, it has to be noted that its duty is to "feed" the sluice, and that unless the channel is capable of conveying to the sluice, at all times, the full amount of the discharge, the result must be a loss of efficiency. Thus the channel should be made large enough to carry the maximum discharge q_o. In the present example, working on the *surface* slope of 6 inches per mile, and using Kutter's coefficient of roughness $N={\cdot}025$, the required discharge of 1,025 *c.f.s.*, would be given by a channel with a depth of 8 feet and a bed-width of about 54 feet.

The effect of velocity of approach is very slight. The "virtual" area of the weir-discharge is $c'_w\,b_w\,D$, that is$={\cdot}62\times39\times4=96{\cdot}72$ square feet; and the area of waterway in the channel is $(54+8)\ 8=496$ square feet. Thus the index-value of $\frac{ca}{\Omega}$ for use in Table X is $\frac{97}{496}={\cdot}195$, and the corresponding value of C_w is about 1·02. This small increase of efficiency does not necessitate any alteration in the design.

8. The equation of discharge of an open channel, when the surface is not parallel to the bed, is necessarily a complicated one, and it will scarcely be feasible in the limits of this paper to trace the method by which it is obtained, beyond simply mentioning that it is the equation of fluid motion in the "Eulerian" form. First it is necessary to quote the ordinary equation of flow when the surface of the stream is parallel to the bed of the channel. With the notation used in this paper it is

$$q = wu = w\sqrt{\frac{2gmi}{\zeta}} \quad . \quad . \quad . \quad . \quad (37)$$

Where q is the discharge; u the mean velocity; w the area of cross-section of the stream; m the hydraulic mean depth; i the sine of the angle of inclination of the bed of the channel; and ζ the coefficient of friction. The formula is used in this form by D'Arcy and Bazin. In what is usually known as the "Chezy" formulæ, a single symbol is used to represent the quantity $\sqrt{\frac{2g}{\zeta}}$, thus

$$q = wu = wC\sqrt{mi} \quad . \quad . \quad . \quad . \quad (38)$$

This symbol is the coefficient of velocity, and it is important to recognise the relation between the coefficients of friction and of velocity, which is

$$C = \sqrt{\frac{2g}{\zeta}}\ ;\ \zeta = \frac{2g}{C^2} \quad . \quad . \quad . \quad . \quad . \quad (39)$$

This relation has to be employed if we wish to use Kutter's coefficients in the following results, in which the coefficient of friction ζ is employed. The formula can be put in another form, which will be used in these results, by writing the full value of m, *viz* $\frac{w}{p}$ where w is the area of cross-section and p the wetted perimeter. Then it becomes

$$q = w\sqrt{\frac{2gwi}{\zeta p}}\ ;\ \text{or}\ q^2=\frac{2gi}{\zeta p}\,w^3 \quad . \quad . \quad (40)$$

To form the differential equation of motion we have to consider the changes which the external forces acting on the fluid will produce in the momentum of the fluid contained in a definite region of space, *i.e.*, between two fixed imaginary cross-sections. To do this we have to equate (*i*) the net rate of in-flow of momentum into the region, *plus* (*ii*) the external forces acting on the fluid in the region, to (*iii*) the rate of change of momentum in the region, *i.e.*, to *zero*, since the motion is "steady." The equation is as follows :—

$$giw^3-\frac{\zeta}{2}pq^2-\frac{g}{b}w^3\frac{dw}{dx}+q^2\frac{dw}{dx}=0\,. \quad . \quad . \quad . \quad (41)$$

The meaning of the notation is as follows :—

x=distance of section under consideration from the origin. The axis of x coincides with the bed of the channel, and the origin is at the place where the surface becomes parallel to the bed of the channel, *i.e.,* at an infinite distance upstream. In the result the integration extends between the limits x_1 and x_2. The section at x_2 is situated at the lower end of the stream, at the weir or fall which causes the absence of parallelism between the surface and the bed, while the section at x_1 is at a distance L upstream, so that $x_2-x_1=L$.

p=wetted perimeter of stream.

b=surface width of stream.

w=area of cross-section of stream. This is taken as the dependent variable.

i=longitudinal inclination of bed.

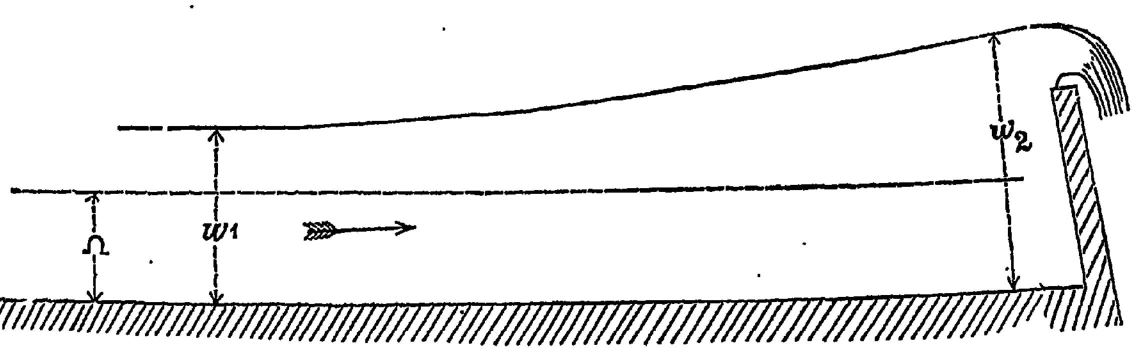
ζ=coefficient of friction.

To integrate equation (40), it can be put in the following form, where a^3 stands for $\frac{q^2}{g}b$, and Ω^3 for $\frac{\zeta q^2}{2gi}p$:—

$$i\,dx=\frac{w^3-a^3}{\Omega^3-w^3}\frac{dw}{b} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (42)$$

Now, comparing the value of Ω with (40), it is seen that Ω is the area of cross-section that would result in the discharge q, if the surface of the stream were parallel to the bed. Taking as limits w_2, the cross-section at the weir or fall, and w_1 the cross-section at a distance L upstream, the final result, for a stream whose depth is *increasing* in the direction of flow, is as follows (*see* Fig. 14) :—

Fig. 14.



$$\frac{bi\,L}{\Omega} = \frac{w_2}{\Omega} - \frac{w_1}{\Omega} + \frac{1}{6}\left(1 - \frac{2i}{\zeta}\frac{b}{p}\right)F_1 \quad . \quad . \quad . \quad . \quad . \quad (43)$$

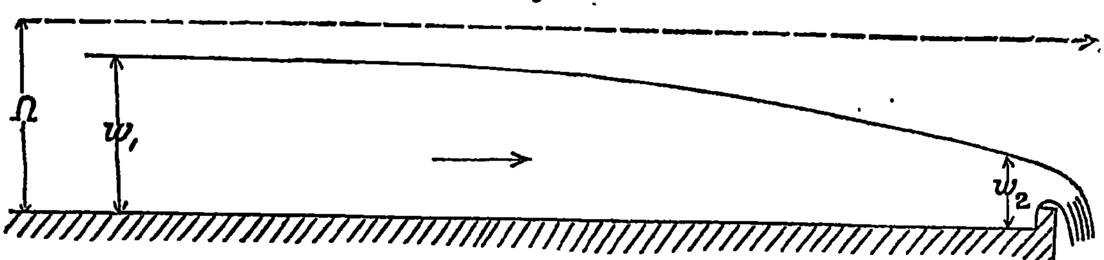
where the value of F_1 is

$$F_1 = \log_e\left\{\frac{\left(\frac{w_1^3}{\Omega^3} - 1\right)\left(\frac{w_2}{\Omega} - 1\right)^3}{\left(\frac{w_1}{\Omega} - 1\right)^3\left(\frac{w_2^3}{\Omega^3} - 1\right)}\right\} - 2\sqrt{3}\left\{\tan^{-1}\left(\frac{1}{\sqrt{3}} + \frac{2}{\sqrt{3}}\frac{w_2}{\Omega}\right)\right.$$
$$\left. - \tan^{-1}\left(\frac{1}{\sqrt{3}} + \frac{2}{\sqrt{3}}\frac{w_1}{\Omega}\right)\right\}$$

The arithmetical values of F_1 for various values of the ratios $\frac{w_1}{\Omega}$ and $\frac{w_2}{\Omega}$ are shown in Table XI * :—

For a stream which is *decreasing* in depth in the direction of flow, the result is as follows (*see* fig. 15) :—

Fig. 15.



$$\frac{bi\,L}{\Omega} + \frac{w_1}{\Omega} - \frac{w_2}{\Omega} = \frac{1}{6}\left(1 - \frac{2i}{\zeta}\frac{b}{p}\right)F_2 \quad . \quad . \quad . \quad . \quad . \quad (44)$$

Where the value of F_2 is

$$F_2 = \log_e\left\{\frac{\left(1 - \frac{w_1^3}{\Omega^3}\right)\left(1 - \frac{w_2}{\Omega}\right)^3}{\left(1 - \frac{w_1}{\Omega}\right)^3\left(1 - \frac{w_2^3}{\Omega^3}\right)}\right\} + 2\sqrt{3}\left\{\tan^{-1}\left(\frac{1}{\sqrt{3}} + \frac{2}{\sqrt{3}}\frac{w_1}{\Omega}\right) - \right.$$
$$\left. \tan^{-1}\left(\frac{1}{\sqrt{3}} + \frac{2}{\sqrt{3}}\frac{w_2}{\Omega}\right)\right\}.$$

The arithmetical values of F_2 are shown in Table XII.

This result, unfortunately, involves the use of trial methods, but with the help of the tables the calculations are not prohibitively cumbrous. The values of b and p will vary somewhat throughout the length under consideration, but, as a rule, the variation will not be very much, and the arithmetical mean of the greatest and least values may be used. There is also some small variation in the value of ζ, but this quantity occurs in a term which has a very small influence on the result. A *trial* value of ζ has to be used first and, if necessary, the result can be corrected afterwards, but it will not, as a rule, be necessary to do so.

As an example take the case of a canal whose bed-width is 100 feet; side-slopes 1 to 1; bed-slope 1 per 1,000, or very nearly 6 inches per mile; and length 4 miles. The depth of water at the head is 7 feet and at the tail 3 feet; find the discharge. In this case q is not known, so ζ is indeterminate, and a trial value must be used. The depth increases upstream, and the normal depth, with surface parallel to bed, would exceed 7 feet, the depth at the head of the canal. Assume that the normal depth will be 8 feet. Then (from Jackson's Tables, pp. 292-293) the value of q is 1,967 and of c is ·858. The value of ζ is thus $\frac{64{\cdot}4}{(85{\cdot}8)^2}$, that is, ·0087478. Then, $\frac{2i}{\zeta} = \frac{2 \times {\cdot}0001}{{\cdot}0087478} = {\cdot}022863$. Next, as regards b and p, the surface-width at a depth of 7 feet is 114 feet and at a depth of 3 feet b is 106 feet. The variation is small, and the mean value 110 may be used. The value of p is 119·8 feet at the head and

*Not printed.

108·5 feet at the tail. We can take the mean 114 feet. Thus the factor $\frac{1}{6}(1-\frac{2i\,b}{\zeta\,p})$ becomes $\frac{1}{6}(1-\cdot022061)$ that is, ·16299. The values of w_1 and w^2 are 749 and 309 square feet respectively, and equation (44) becomes

$$\frac{110 \times \cdot0001 \times 4 \times 5280}{\Omega} + \frac{749}{\Omega} - \frac{309}{\Omega} = 16299\ F_2$$

That is,

$$\frac{672}{\Omega} = \cdot16299\ F_2;\ \text{or}\ F_2 = \frac{4123}{\Omega}\quad .\ .\ .\ .\ .\ .\quad (45)$$

where F_2 depends on the values of $\frac{w_1}{\Omega}$ and $\frac{w_2}{\Omega}$. Trial values of Ω have to be selected until one is found to satisfy (45).

First, try $\Omega = 800$. Then $\frac{w_1}{\Omega} = \cdot93625$ and $\frac{w_2}{\Omega} = \cdot38625$: and from Table XII the value of F_2 is found to be 5·9510. Then the trial equation (45) is

$$F_2 = \frac{4123}{800} = 5\cdot1538$$

A result which does not correspond with the value from Table XII.

Next try $\frac{w_1}{\Omega} = \cdot80$; $\frac{w_2}{\Omega} = \cdot33$.

Then $\Omega = \frac{749}{\cdot8} = 932$, and F_2 (from Table XII) is 3·703.

From equation (45) $F_2 = \frac{4123}{932} = 4\cdot4237$.

As a third trial take $\frac{w_1}{\Omega} = \cdot86$; $\frac{w_2}{\Omega} = \cdot355$

Then $\Omega = \frac{749}{\cdot86} = 871$, and F_2 (from Table XII) is 4·4200.

From (45), $F_2 = \frac{4123}{871} = 4\cdot74$.

Again, try $\frac{w_1}{\Omega} = \cdot89$; $\frac{w_2}{\Omega} = \cdot367$.

Then $\Omega = \frac{749}{\cdot89} = 842$, and F_2 (from Table XII) is 4·8962.

From (45), $F_2 = \frac{4123}{842} = 4\cdot8991$.

This is exact.

The value of Ω being 842 square feet, the depth of the canal is 7·81 feet. In calculating ζ we took the depth as 8 feet, and this small error of ζ will not affect the result. The value of q, by reference to Jackson's Tables, is, by proportional parts, 1,885 c. f. s.

Table XII.

Table showing values of the factor F_2; where

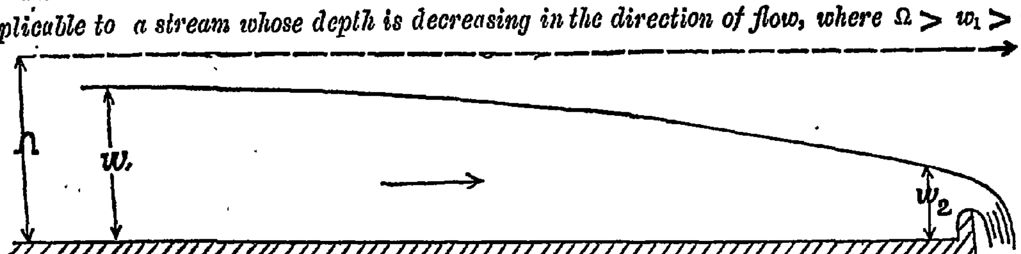
$$F_2 = (L_1 + T_1) - (L_2 + T_2)$$

and

$$L_1 = \log_e\left\{\frac{1-\left(\frac{w_1}{\Omega}\right)^3}{\left(1-\frac{w_1}{\Omega}\right)^3}\right\};\qquad L_2 = \log_e\left\{\frac{1-\left(\frac{w_2}{\Omega}\right)^3}{\left(1-\frac{w}{\Omega}\right)^3}\right\}$$

$$T_1 = 3\cdot464\ \tan^{-1}\left(\cdot57736 + 1\cdot15472\frac{w_1}{\Omega}\right);\quad T_2 = 3\cdot464\ \tan^1\left(\cdot57736 + 1\cdot15472\ w^2\right)$$

Applicable to a stream whose depth is decreasing in the direction of flow, where $\Omega > w_1 > w_2$



·09	·07	·05									
·8023	·7823	·9024									
·14	·12	·09	·05								
·0654	·7856	·9058	1·2059								
·18	·15	·12	·09	·05							
·7303	·0120	1·0920	1·2722	1·5123							
·18	·15	·12	·09	·05							
1·0415	1·227	1·4027	1·5889	1·6280							
·23	·20	·15	·10	·05							
1·0567	1·2375	1·5390	1·8300	2·0309							
1·25	·20	·15	·10	·05							
1·2590	1·5625	1·8046	2.1046	2·4040							
·25	·20	·15	·10	·05							
1·6952	1·8087	1·2008	1·5008	1·8011							
·30	·25	·20	·15	·10							
1·6395	1·9559	2·2404	2·5515	2·8515							
·30	·25	·20	·15	·10							
2·0104	2·3169	2·6203	2·9224	3·2:24							
·30	·25	·20	·15	·10							
2·4078	2·7142	3·0177	3·3109	3·6198							
·35	·30	·25	·20	·15	·10						
2·6309	2·8410	3·1450	3·4515	3 7530	4·0530						
·40	·35	·30	·25	·20	·15	·10					
2·6907	3·0106	3·3273	3·6337	3·9372	4·2303	4·5393					
·45	·40	·35	·30	·25	·20	·15	·10				
2 9377	3·2617	3·5796	3·8003	4·1907	4 5002	4·5023					
·50	·45	·40	·35	·30	·25	·20	·15	·10			
3·2603	3·6200	3·9510	4·2070	4·5760	4·6850	5·1995	5 4906				
·55	·50	·45	·40	·35	·30	·25	·20	·15	·10		
3·8559	4·3006	4·5458	4·8708	5·1877	5·4084	5·8049	6·1093	6·4104	6·7104		
·55	·50	·45	·40	·35	·30	·25	·20	·15	·10		
4·6351	4·9904	5·3220	5·6476	5·9615	5·2752	6·5816	6·8851	7·1672	7·4872		
·60	·55	·50	·45	·40	·35	·30	·25	·20	·15	·10	
4·9791	5·3503	5·7010	6·0373	6·3632	6·6791	6·9809	7·2003	7·5997	7·9016	8·2018	
·60	·55	·50	·45	·40	·35	·30	·25	·20	·15	·10	
6·0123	6·4132	6·7630	7·1001	7·4251	7·7420	8 0537	8·3501	8·6020	8·9647	9·2617	
·65	·60	·55	·50	·45	·40	·35	·30	·25	·20	·15	·10
6·4704	6·8738	7·2447	7·6054	7·9310	8·2500	8·5785	8·8913	9·1006	9·4041	9·7062	10 0963
·65	·60	·55	·50	·45	·40	·35	·30	·25	·20	·15	·10
7·6820	8 2800	8·6500	9·0016	9·3378	9·6028	9·9707	10·2004	10·5008	10·9003	11·2034	11·5024

PAPER No. 26.

Increase of torrent-floods in the upper reaches of the Eastern Jumna Canal.

As the Government are now considering many irrigation projects which entail bold schemes for crossing drainage, I think a discussion on the above subject will be profitable.

Plate No. 48 exhibits the torrents which cross the Eastern Jumna Canal in the first 17 miles of its course.

Above Naushera weir and regulator, at mile 6, we have had no recent troubles worthy of notice.

At mile 8½ the Chapri *rau* is encountered. This channel is estimated to carry 14,000 cusecs, but I may here remark, that torrent-discharges are very doubtful, as the water comes down in huge waves, and debars calculations approaching to any degree of accuracy. When I was Assistant Engineer on the Eastern Jumna in 1877 to 1880, the Chapri was allowed to pour into the Canal, and find egress at Naugaon weir, mile 13½. A big flood did not stick to the Canal, but burst out at convenient points.

The waterway at Naugaon consisted of 12 openings, of 10 feet span. Originally it was a level crossing, but from the accumulation of floods at this point, the bed became so scoured out on the downstream, that the work gradually assumed the form of a weir-rapid 20 feet high with a slope of 1 in 7.

In July 1880, just as the rice crop irrigation was in full swing, the three torrents met at Naugaon, overflowed the bank, and cut out a huge chasm 300 feet long and 30 feet deep.

By good fortune a break of a fortnight followed, and in that time the breach was made up, and no crops were lost. Immediate arrangements were made to get up a scheme for diverting the Chapri down a channel of its own; but, though many projects were submitted, want of funds prohibited all active measures until 1889, when the Raipur regulator and escape was sanctioned. It was completed in 1890 and cost Rs. 75,000.

A regulator was thrown across the Canal, and an escape to the Jumna was excavated, with a bed-width of 80 feet.

These measures were supposed to have secured the Canal, but in 1894 the floods proved again too large for Naugaon weir. The whole work nearly fell in, and was only saved by cutting the banks and throwing in 1½ lacs of cubic feet of heavy material to check the scour.

Projects were then immediately prepared for extending the opening and providing a subsidiary weir on the downstream.

These measures cost Rs. 1,70,000, and increased the waterway 50 per cent., by adding six 10-foot openings to the original twelve.

Matters were then thought secure, but in 1901 the floods again proved too great for each of the three outlets at Raipur, Naugaon, and Muskurra.

Raipur regulator was submerged by a 14-foot flood, and the water not only filled the escape, but went over the regulator and sent a torrent down the Canal.

Naugaon revetments were just overtopped, but no accident occurred. Muskurra revetments were also topped, but the staff at site saved the position with bricks and boulders.

We have now broadened the Raipur escape from 80 feet to 120 feet, which, I hope, will prevent further overflow towards Naugaon.

This measure cost Rs. 26,000. The revetments at Naugaon have been raised 2 feet, and the banks cut down at convenient points so as to provide safety valves. The Muskurra revetments have also been raised and the dam cut down 2 feet. The new crest is equipped with falling gates 4 feet high, which had a most valuable effect in removing shoals, and leaving the weir in an unmasked condition. I place great weight on this latter point, and I think it has probably doubled the discharging power of the weir.

The table below gives a useful comparison of gauges and shows that an 8-foot torrent has been converted into one of 6·5. As these measurements were made above the weir, the results are most satisfactory.

Table of Gauge.

Date.	Gauge, Kalsia Regulator.	Gauge, Jauli Dam.	Remarks.
21st June 1899	7·4	1·3	The Kalsia Regulator gauge represents depth of water on crest of Muskhara Dam, and Jauli Dam gauge the depth on its crest. The Muskhara Dam was masked by a projecting shoal which was removed in early part of 1902.
3rd July 1899	9·0	3·0	
21st ditto	7·4	1·8	
25th July 1900	7·9	1·9	
29th ditto	8·0	2·2	
28th August 1900	8·0	1·8	
30th ditto	8·2	1·85	
18th July 1901	9·2	2·9	
3rd August 1901	11.2	4·3	
22nd ditto	10·3	3·7	
30th ditto	7·6	2·9	
18th July 1902	8·5	3·1	
21st ditto	6·5	2.2	
21st July 1903	6·5	2·3	
5th August 1903	5·0	2·5	

At present it may be deemed that the discharging power of the three outlets is equal to such floods as have hitherto occurred. But if the torrent-volumes go on increasing, as hitherto they have done, so must the engineers devise additions to this weir. For the last twenty years the Forest Department have been furnishing us with reports on the fire protection, and have endeavoured to prevent denudation. Some good has been done by these measures, but it is hard to remedy the neglect of a half-century.

Part of the catchment lies within forest bounds, and here we are fairly safe from further denudation. Outside the line the case is, I am afraid, not so satisfactory. The cultivators are a hapless lot, and prefer to grow a poor class of kharif crop which does not require much tillage or much moisture in the ground. Consequently their fields consist of large sloping plots, from which the rainfall rapidly runs off, and in some cases has denuded the ground to the bare rock. When this occurs, a new bit of jungle is broken up, and such action must of course increase the torrential discharge.

One officer has suggested, that we ought to take up the land, turn it into a forest preserve or grass farm, and give the inhabitants plots elsewhere. But I fear the Revenue Department would not listen to this. In a similar case on the Upper Ganges Canal, the Lieutenant-Governor gave me permission to lease a village, and negotiations were opened with the Collector, who has, however, reported the scheme as impossible. My object was to sub-let it to the present tenants, assist them in making "kiaries," and start an improved form of cultivation. The scheme looked promising, but at present there is a deadlock on account of difficulties in dealing with innumerable co-sharers.

On the Eastern Jumna, Mr. Laurie is experimenting on the catchment of a small torrent near Kalsia, which may lead to something. He has got leave from the cultivators to put up the field bunds during this monsoon, and thus endeavour to make the system popular. Whenever, however, they sow *juar*, *cheri*, and maize, they will of course cut the bunds to stop swamping.

My object in writing this paper is to show the great danger of torrent works which do not give ample water-way, and do not allow for the probablity of the discharges increasing.

As regards design, it seems the best policy is not to depend on computed volumes, as is done in the plains, but to examine the channel widths and work from them. Super-passages such as Cautley provided at Pathri and Ranipur have many advantages, but the conditions of every case will not always allow for such designs. Quite recently I wished to press the advantages of a steel syphon against the submitted design of an aqueduct for a small canal over a torrent in the Tarai. But I gave way, as the Executive Engineer pointed out that a previous syphon had failed, on account of being choked by a gravel deposit. I thought of overruling this plea on the ground that a high velocity could be created so as to stop such an eventuality. This idea was, however, dropped, as it was feared that the presence of gravel and a swift flow would certainly abrade the metal, and give it a short life.

PAPER No. 27.

The Mat Branch Extension, Ganges Canal.

In Colonel Cautley's report on the Ganges Canal, he proposed to make a channel from mile 110 of that Canal, to convey water into the Aligarh and Muttra districts through several intricate branches. This was first termed the Bulandshahr Branch, but the name was afterwards altered to the Mat Branch. From 1851 to 1855 a supply channel was made, 10½ miles long, which then bifurcated into the left branch or Barauda distributary, and the right or Mat branch, the latter 41 miles in length.

In 1874 an estimate amounting to Rs. 7,00,000 was sanctioned to extend the Mat branch into the Muttra district, and work was begun on the channel during the famine of 1877-1878. The proposal for the extension was abandoned in 1879, when it was finally settled that the tract of country to be served was then securely protected by wells in ordinary years, and fairly protected in dry years, as water was only between 35 and 45 feet below the surface.

During the next 20 years this tract underwent a remarkable transformation. Spring level subsided from 18 to 20 feet, and a large number of wells ran dry. The causes of this subsidence were; (i) a series of years of light rainfall; (ii) the improvement of the channel of the Karwan nadi, which is the main line of drainage in this part of the country; the diversion of much of the drainage from the Karwan nadi into the river Jumna; (iv) the construction of three large drains in the Bulandshahr district and three others in the Aligarh district; and (v) the shrinkage in the river Jumna caused by large supplies drawn off by the two Jumna Canals and by the opening of the Agra Canal.

The result of this fall in spring level was that 46 per cent. of the wells in the tract between the river Jumna and the Karwan nadi became brackish and useless for germinating seed; mortality increased, wholesale emigration took place, and a troublesome weed named *bansurai* (*pluchea lanceolata*) spread to a very injurious extent. In consequence of these evil effects, revenue collections fell off by 40 per cent. and in April 1901 the Board of Revenue advocated the introduction of canal-water into this tract; whereupon Mr. C. G. Palmer, C.I.E., Chief Engineer, ordered the preparation of a project for the extension.

A project for the Mat Branch extension was prepared from May to September 1901, and was sanctioned in June 1902 against open Ganges Canal capital at a cost of Rs. 13,48,043 under the following heads:—

	Miles.	Rs.
2. Main line, widening supply channel	10½	22,124
Alterations, Mat Branch	41	2,20,242
Main line extension, 39½ miles, and escapes, 6 miles	45½	2,97,212
Total	97	5,39,578
3. Distributaries	222	3,91,801
4. Drainage works	26	12,979
Total works	345	9,44,358
Establishment, tools and plant		2,16,035
Total direct charges		11,60,393
Indirect charges		1,28,400
Interest charges		59,250
GRAND TOTAL		13,48,043

Five years from the time the estimate was sanctioned were allowed for the construction of the extension, but the work was so rapidly carried out that it was practically finished within a year of being started, and all the accounts for the work will be closed at the end of the present financial year, 1904-1905. The extension was only meant to be opened in April 1905, but was actually opened in December 1903, and in that season, 26,908 acres of rabi irrigation was effected. The accounts will be also closed within $2\frac{3}{4}$ years after sanction was first conveyed to the estimate.

The Mat Branch has been designed to carry 916 cusecs at the head, of which 434 cusecs are meant for the irrigation from old channels of the branch. This in 1899-1900 amounted to 31,806 acres in the kharif, and 63,338 in the rabi, making a total of 95,144 acres, with duties of about 73 and 144 acres respectively, and 482 cusecs for irrigation from the extension, which it is estimated will in a year of drought reach a maximum of about 36,000 acres in the kharif and 74,000 in the rabi, making a total of 110,000 acres, with duties of 75 and 150 acres respectively, when irrigation is fully developed on the extension in course of time.

At the end of the rains of 1902 preliminaries for the work were begun village maps were prepared, lines levelled and lock-spitted, and the establishment selected for carrying it out. The portions including widening the supply channel, alterations to the Mat Branch, and making 37 miles of distributaries, a total of 89 miles of channel, were made a special sub-division in December 1902, in charge of Rai Sri Ram Bahadur, Assistant Engineer, working under Mr. G. T. Anthony, Executive Engineer of the Bulandshahr division.

The remainder of the work, including 45 miles of new main canal and escapes, 185 miles of distributaries, and 26 miles of drains, making a total of 256 miles of channel, was placed in a special division under Mr. H. Nelson, Execuitve Engineer, who formed it in the middle of November 1902, with Lala Kanhaiya Lal and Mr. O. H. Townsend, Assistant Engineers, in charge of the two subdivisions. All these gentlemen set to work with marked zeal and assiduity, with such excellent results that irrigation was carried out at the tail of the Canal, at a distance of 91 miles from the parent stream, within a twelvemonth of work being started.

Since the commencement of preliminaries for the work, Mr. H. Marsh, C.I.E., Chief Engineer, has given every detail of it his closest attention, and his revision of the designs has resulted in their greater efficiency and economy. In November and December 1902 he inspected and approved all lines for the channels, and has given valuable assistance with every portion of their construction.

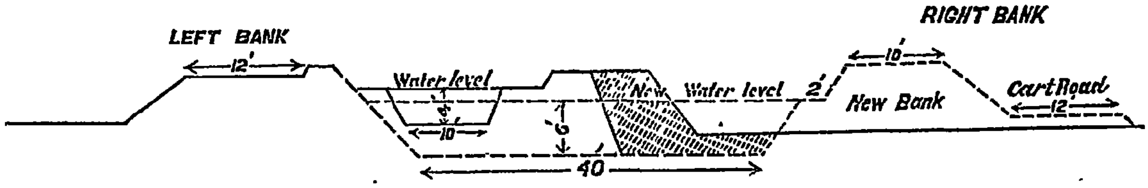
For the preparation of materials, fuel was procurable on the banks of the existing Mat Branch Canal, but little or none was procurable elsewhere: it was therefore necessary to import coal by rail, to burn bricks and lime for the extension, for which two stations of the Bombay, Baroda and Central India Railway were conveniently situated. Good dark nodular kunkur was found everywhere, from which excellent hydraulic lime was made; it was burnt in intermittent kilns and pulverized in Lucop distintegrators, at a cost of Rs. 21 per 100 cubic feet. Some large hard block kunkur was found near the upper part of the extension, but elsewhere it was small and soft. Good clay for bricks and puddle was also procurable everywhere. Bricks, $9\frac{3}{4}\times4\frac{3}{4}\times2\frac{3}{4}$ inches in size, were burnt in Bull's kilns, and turned out at rates of Rs. 10-8-0, Rs. 7-8-0, and Rs. 4-0-0 for the three classes. Washed nodular kunkur, or brick ballast, was used for ballast. Allahabad tiles were procured from Aligarh: ironwork, teak doors, and windows from Bombay; stone mile, furlong, and boundary pillars from the Dholpur State quarries.

The widening of the supply channel was quickly carried out during the rains of 1903 when the Canal was closed; a right bank cart-road was made with the spoil, the left bank being retained as a driving road. This channel was formerly 40 feet wide, and discharged 420 cusecs with a depth of 5 feet, and slopes of from 2 to 1·2 foot per mile. It was widened to 60 feet to discharge 916 cusecs with a depth of 5·2 feet, gradually increasing to 5·7 feet, but, owing to accumulations of silt in the bed, a gauge of 5·2 feet only gives a discharge

of 742 cusecs at present, and one of 5·7 feet has to be run at the head to get in full supply; this higher gauge also assists to scour out the silt lying on the bed—a process which is going on rapidly at the lower end. Planks are always placed in the head groves to maintain a head of 0·3 foot, and to admit only clear surface water into the channel.

The masonry works of the supply channel consist of a head with 8 spans of 5½ feet, 2 bridges with 4 spans of 11 feet, an aqueduct with 2 spans of 24 feet, 3 bridges with single spans of 45 feet, and another with 2 spans of 22½ feet. No alterations were required for any of these works, except side-pitching, both upstream and downstream, for all the bridges and the aqueduct, and a small extension of the siphon at mile 10 on the right side for the cart-road. The estimate for this portion of the work amounted to Rs. 22,124, but the work was actually carried out at a cost of only Rs. 12,360.

Alterations to the existing Mat Branch were difficult to carry out because the channel had to be run whenever there was any demand for irrigation. Consequently during the first seven months it was only possible to make the distributaries, to excavate the channel above water level in the first 9 miles and in short lengths elsewhere, and to make up the outer bank with spoil from part of the existing right bank in the remaining 32 miles. The last was done by taking away the portion of the left bank hatched as follows:—



Old work – Full lines   *New work – dotted lined.*

During the rains closure, the remainder of the excavation was carried out to complete the new outer bank, and to make up the right bank cart-road.

Excavation of the bed and building masonry works on the Mat Branch could not be started till the Canal was closed in the middle of July 1903; the rains continued well into September, and in October an opportune fall of 1 inch staved off demand for rabi irrigation until December; this gave a space of 5 months for carrying out all bed-excavation, for completing the outer bank, and for altering and building the masonry works. The channel of this branch was altered to a slope of 0·64 foot per mile with small falls at bridges, to carry 763 cusecs at the head with a depth of 6 feet and bed-width of 59 feet. In the first 9 miles the bed was only excavated to a construction width of 47 feet until irrigation has developed; elsewhere the outer bank was made for the full bed-width; this necessitated some temporary regulators being afterwards built at bridges and falls, to hold up water to the required level until the channel above is made of the full dimensions.

In this portion of the work 27 new bridges were built of 3 spans varying from 12 to 15 feet, also 2 falls, 6 distributary and minor heads. The following additions and alterations to existing works were made: the head and 8 large bridges were heavily pitched, and a new floor was made for the railway bridge; the spans of 5 bridges of 15 and 16 feet were doubled; 4 unnecessary siphons were removed, 6 siphons were lowered and the barrels lengthened, 1 was only lowered, and 4 siphons had their barrels lengthened; one fall had its water-way doubled, another fall was altered and heavily pitched; and 2 escape-heads were altered. This all comprised the building of 35 new works and additions and alterations to 34 existing works.

Alterations to the Mat Branch were carried out at a cost of Rs. 2,19,470 against an estimated amount of Rs. 2,20,242. The quantities of work and the average rate at which it was done is shown below :—

			Rs.
Earthwork, dry,	46,148,758	c. ft. at	2·7
" wet,	105,627	"	10·5
Concrete,	71,101	"	20·1
Brickwork,	159,971	"	24·1
Archwork,	27,890	"	33·0
Plaster,	15,563	"	3·1
Pitching,	114,170	"	4·7
Metalling,	23,021	"	4·0
Demolition,	104,356	"	1·9

This is irrespective of work on 89 miles of distributaries, which is included under figures for the extension distributaries. The rate for pitching was low, as old materials were chiefly used.

From mile 41 to mile 81 the extension was made as an entirely new channel, except where some of the excavation of 1878 had not been filled in. It was made with a uniform slope of 0·64 foot per mile, with numerous small falls, to carry 446 cusecs at the head with a bed-width of 38 feet and a depth of 6 feet of water, reduced to 77 cusecs at the tail with a bed-width of 18 feet and a depth of 3·6 feet. The masonry works in this reach comprised 7 inspection houses, 43 bridges, including 1 railway bridge, 1 regulator, 5 falls, 2 siphons, and 8 distributary heads, making a total of 66 works. All this was done at a cost of Rs. 2,82,790 against an estimate of Rs. 2,97,212.

The quantities of work and the average rates at which it was done on the Mat Branch extension are as follows :—

			Rs.
Earthwork,	44,055,502	c. ft. at	2·6
Concrete,	95,272	"	21 5
Brickwork,	184,901	"	29 7
Archwork,	25,266	"	34·9
Plaster,	58,537	"	4·2
Pitching,	84,246	"	13·3
Metalling,	24,426	"	5·8

The total length of distributaries excavated amounted to 222 miles, on which were built 178 bridges, 3 of which were railway bridges, 2 regulators, 87 falls, 2 siphons, and 9 heads for minors. This work was done at a cost of Rs. 3,60,413 against an estimated amount of Rs. 3,91,801. Two more small channels are to be made to irrigate some distressed portions of the Aligarh district towards Mursan, and a third channel is contemplated to carry water towards Hathras, which will be constructed in the near future.

The quantities of each sort of work in making these distributaries and the average rates paid were as follows :—

			Rs.
Earthwork,	64,487,582	c. ft. at	2·1
Concrete,	133,837	"	21·3
Brickwork,	204,389	"	29·5
Archwork,	30,925	"	34·6
Plaster,	27,893	"	3·9
Pitching,	44,825	"	10·7
Metalling,	65,523	"	5·3
Demolition,	28,872	"	1·4

Owing to the scarcity of water in the Muttra district, it was found necessary, at the very beginning of the work, to excavate a small boundary ditch channel from mile 41, at the tail of the Mat Branch, up to mile 81, the tail of the extension, in order to provide water for drinking purposes, also to moisten the ground where hard, to facilitate excavation, for the manufacture of bricks, and the preparation of materials. All surplus water was used for irrigation, and it is noteworthy that 432 acres of kharif irrigation was actually done in 1903

within six months of starting work; this was included in the figures of the Bulandshahr division. The cost of this supplementary channel was Rs. 15,184—a large sum—but it well repaid the expenditure incurred, and portions of it have already been utilized for boundary minors, which are doing excellent work at present.

There are now two escapes on the Mat Branch, an old and a new one. The old Kot escape into the Hindan nadi has its head at the end of the supply channel, just above the Mat head, and discharges a maximum of 644 cusecs; this is one of the three chief escapes of the Ganges Canal. A new Harnaul escape was made from mile 58 of the extension, and tails into the Jumna with a channel 4 miles long; the full discharge of this escape is 350 cusecs, which is the whole volume carried by the Canal at mile 58. Four small escapes of a total length of 2 miles were also made from distributaries, and discharge altogether 65 cusecs.

Only one drain, four miles long, was proposed and has been made at a cost of Rs. 1,757. Provision has been made in the estimate for 22 miles of additional drainage cuts, which will be made when found necessary: at present no further drains seem to be required, for there are no swamps in the country served by the extension, spring level is from 65 to 80 feet below ground-level, and it would be a benefit if it were raised to replenish dry wells. Seven inspection houses have been built on the extension—four on the main Canal and three on distributaries: two ziladar's offices, a dispensary, and several beldars' huts are also in course of construction.

The total expenditure against the Mat Branch extension estimate of Rs. 9,44,358, for works alone, has been Rs. 8,32,807 under the three heads of (2) main Canal, including all works on the supply channel, altering the Mat Branch, and the extension proper, (3) distributaries, and (4) drainage works. The total quantity of work carried out against this expenditure was the following:—

	c. ft.
Earthwork, dry	158,521,375
„ wet	105,627
Concrete	300,463
Brickwork	549,711
Archwork	84,081
Plaster	102,029
Pitching	243,241
Metalling	112,970
Demolition	146,631

The financial forecast showed that the extension would probably commence to earn revenue in 1905, and irrigation would be fully developed in 1937 after the Canal has been working for 32 years, when the net receipts would probably amount to 18·7 per cent. on the outlay incurred in making it. These figures are likely to be reached even more rapidly than in the forecast, because revenue has been aleady earned in the present year, and, as most of the water-courses have been excavated for the cultivation, irrigation is likely to be fully developed in about 16 years.

Schedule of masonry works in altering existing Mat Branch.

Position.		Description of work. Regulators.	Remarks.
Mile.	Feet.		
...	...	Head and regulator	No alteration. The old regulator works without trouble.
20	1,370	Rakhera	Removed, because not wanted.
23	2,480	Garhi	Ditto ditto.
		E.—Falls.	
9	500	Sharakpur fall	V notch removed and grooves put in instead.
17	3,795	Inaitpur „	Another 12 foot span added to the old fall, but not yet arched.
18	1,320	Kapna „	A new fall built.
27	3,960	Kalakhuri „	Ditto.

Schedule of masonry works in altering existing Mat Branch—continued.

Position. Mile.	Feet.	Description of work, Regulators.	Remarks.
		F.—Other cross drainage works.	
6	660	Drainage siphon . .	Siphon lowered.
10	1,920	Ditto . .	Old siphon extended in length.
13	3,300	Ditto . .	Ditto ditto.
15	1,320	Ditto . .	Ditto ditto.
20	2,310	Ditto . .	Old siphon lowered to suit the new bed level and extended in length.
24	3,360	Ditto . .	Ditto ditto ditto.
26	990	Ditto . .	A 12-inch pipe siphon removed.
28	4,840	Ditto . .	An old siphon not in use removed.
28	4,860	Ditto . .	Old siphon lowered and extended to suit the new bed, and an escape head built over it.
32	...	Ditto . .	Old siphon lowered and extended to suit the new bed.
33	400	Ditto . .	Old siphon extended in length, and an escape head built over it.
35	4,020	Road siphon . .	A village road bridge built here instead.
37	2,700	Drainage siphon . .	Old siphon lowered and extended to suit the new bed.
38	3,880	Tappal road siphon .	A first class road bridge built here.
39	1,320	Drainage siphon . .	Old siphon lowered and extended to suit the new bed.
		G.—Bridges.	
...	60	Grand trunk road bridge .	No alteration to the bridge. Slopes pitched to suit new bed.
2	180	Basantpur bridge . .	Ditto ditto ditto.
2	5,115	Daula bridge . .	Ditto ditto ditto.
4	1,100	Village road bridge . .	Ditto ditto ditto.
5	3,040	East Indian railway bridge	No alteration to the bridge. Curtain walls built, and block kunkur floor put in.
5	4,950	Jamalpur bridge . .	No alteration to the bridge. Slopes pitched to suit new bed.
7	1,815	Bilaspur " . .	Ditto ditto ditto.
8	2,145	Banjarpur " . .	Ditto ditto ditto.
10	2,970	Foot " . .	New cart-road bridge built, 3 spans of 15 feet.
11	3,145	Sunpera " . .	No alteration, except pitching the slopes.
12	2,640	Foot " . .	New foot bridge built, 3 spans of 15 feet.
13	1,900	Rawani " . .	One span of 16 feet added.
14	220	Foot " . .	New cart-road bridge built, 3 spans of 14 feet.
14	3,460	Village road bridge .	One span of 15 feet added.
15	1,980	Ditto .	Ditto.
16	1,920	Ditto .	Ditto.
17	220	Ditto .	One span of 14 feet added.
18	4,785	Ditto .	New bridge built, with 3 spans of 14 feet each.
19	4,950	Ditto .	Ditto ditto ditto.
21	...	Ditto .	Ditto ditto ditto.
21	2,380	Ditto .	Ditto ditto ditto.
23	1,815	Ditto .	Ditto ditto ditto, crest raised.
23	825	Ditto .	Ditto ditto ditto.
24	685	Ditto .	Ditto ditto ditto.
26	3,300	Ditto .	New bridge built, with 3 spans of 13 feet each, crest raised.
26	825	Ditto .	Ditto ditto each.
27	990	Ditto .	Ditto ditto ditto.
28	4,125	Ditto .	Ditto ditto ditto.
29	2,145	Ditto .	Ditto ditto ditto.
30	500	Ditto .	Ditto ditto ditto, crest raised.
31	825	Ditto .	Ditto ditto each
32	660	Foot bridge . . .	Ditto ditto ditto.
32	1,815	Village road bridge .	Ditto ditto ditto.
33	1,540	Ditto .	New bridge built, with 3 spans of 12 feet each, crest raised.
34	3,465	Ditto .	Ditto ditto each.
36	1,130	Ditto .	Ditto ditto ditto.
37	2,860	Ditto .	Ditto ditto ditto.
38	660	Foot bridge . . .	Ditto ditto ditto.
38	3,270	Ditto . . .	Ditto ditto ditto.
39	3,080	Village road bridge .	Ditto ditto ditto.
40	2,970	Ditto .	Ditto ditto ditto, 2' crest.
		H.—Escapes.	
10	1,920	Hirnoti escape . .	Head altered.
37	1,980	Escape head . . .	Built, head altered.
		O.—Miscellaneous.	
...	1,320	Discharge flume . .	Not built.
...	...	Bed bars . . .	Ditto.
		L.—Earthwork.	
...	...	Earthwork in channel .	45,148,758 cubic feet excavated.

PAPER No. 28.

Damage to the Hindan Dam in 1880.

The Hindan nadi has three main tributaries—the West Kali nadi, which takes its rise near Roorkee, the Nagadeo nadi, with its source in the Siwalik Hills, and the Karsani nadi, which rises near Saharanpur. These three tributaries combine between Barnawa and Meerut to form the Hindan nadi, which joins the river Jumna about 20 miles below Ghaziabad. The whole system is about 130 miles in length, and drains almost all the country lying between the Ganges and Eastern Jumna Canals from the latitude of Saharanpur and Roorkee to below Ghaziabad, having a catchment area of about 2,500 square miles.

The waters of the Hindan nadi are utilized for the Agra Canal, to which they are conveyed by a supply channel known as the Hindan cut, from the Hindan at Ghaziabad to the river Jumna above the Canal head at Okhla. The volume available from the nadi has fallen as low as 126 cusecs, although in floods it amounts to nearly 70,000 cusecs. The minimum combined supply in the Hindan and Jumna for the Agra Canal reached the absurdly low figure of 248 cusecs in the year of drought, 1899. The Agra Canal supplies are, however, augmented by water from the Ganges Canal, which is passed down the Jani escape from Bhola near Meerut, the volume given being limited to a maximum of 1,000 cusecs in the kharif and 900 cusecs in the rabi. Thus it comes about that crops in the Agra district, on the west of the Jumna, are matured with holy water from the river Ganges taken in at Hardwar.

In the final settlement report of the Aligarh district, issued in June 1904, some interesting testimony is adduced by the Settlement Officer to the benefits of canal *versus* well irrigation. In spite of the acknowledged drawbacks to canal irrigation, Mr. W. J. D. Burkitt, I.C.S., does not think that there is any longer a question as to its superiority to well irrigation. One fact, he says, that proves this more than any other is that cultivators, if they can get canal water, will use it in preference to working existing wells. Another proof is the marked greater increase in the rent rates of canal-irrigated land as compared with well-irrigated land. In four canal-irrigated tracts the percentages of increase in the incidence of non-occupancy rentals varied from 91 to 187 per cent.; in three well-irrigated tracts the corresponding percentages were 54 to 72 per cent. It will be quite safe to accept Re. 1 per acre as representing the difference between the letting value of canal and well-irrigated land. He estimates the total increase of assets owing to the Canal in these four canal-irrigated tracts at Rs. 92,764. This estimate may be rather full, but even allowing a margin of error, the balance in favour of the Canal is a large one.

To hold up water in the Hindan nadi and force it into the cut, a dam was built in the nadi below the head of the cut between 1876 and 1878, of the following section (*vide* Fig. 1, Plate No. 49).

The dam was made with 40 bays of 10 feet, and a pitched apron, 10×4 feet, was put in on the right side for 200 feet. The site of the dam was selected at a distance of about 200 feet below the Ghaziabad-Delhi Railway bridge, and about 3 furlongs below the Grand Trunk Road bridge.

In September 1880 some very heavy rain fell over the catchment area of the Hindan nadi; the falls at the chief places are shown below in inches :—

	September 17th.	September 18th.	Total.
Saharanpur	6·0	2·3	8·3
Roorkee	10·7	8·0	18·7
Muzaffarnagar	12·9	4·2	17·1
Meerut	6·8	5·8	12·6

This unusually heavy rain produced an unprecedented flood in the nadi, which gave a gauge of 16·7 feet at the road bridge, 5 spans of 80 feet, with an afflux

of 2·28 feet; a gauge of 15·3 feet at the railway bridge, 6 spans of 75 feet, with an afflux of 1·32 feet; and a gauge of 12·64 feet at the Hindan dam, 40 bays of 10 feet, with an afflux of 1·6 feet.

The discharge of the nadi below the road bridge was calculated to be 67,200 cusecs with a mean velocity of 7·39 feet per sec., but that at the dam was only 50,390 cusecs, because breaches 180 and 280 feet wide were made in the bank, on each side of the road bridge, and a breach 360 feet wide was made in the railway embankment, on the west side of the bridge, through which spills amounting to about 16,810 cusecs of the total volume of the nadi poured over the country, carrying devastation everywhere. The flood in the nadi was described as being like a mountain torrent, advancing in a series of waves, water level in the centre of the stream being from 2 to 3 feet higher than that at the sides. The previous highest recorded flood gauged only 11½ feet at the Grand Trunk Road bridge.

After the flood subsided, an examination of the site showed that very heavy scour had taken place upstream of the dam, to a depth of 16 feet in places; also that three wells in front of piers 15, 16, and 17 had been partly undermined, and the curbs of a large number of other wells had become exposed between piers 12 and 20, 32 and 34, but no settlement of the floor of the dam was found to have taken place. Subsequent investigations showed that a considerable length of the dam floor was lying like a slab, 6 feet thick, the upper end resting on the upstream line of wells acting as pillars, the lower portion lying chiefly on sand, which had got deposited under the dam by the falling river, and on the downstream line of wells the greatest unsupported length being about 26 feet.

The state of the dam after the flood was probably as shown in Fig. 2, Plate No. 50.

The stratum of clay shown in the drawing underlying the upstream line of wells, probably saved the dam from becoming a total wreck.

The unsupported length of floor in each bay, and the depth of scour in front of the bay, are shown in the following table in feet:—

Number of bay.	Unsupported length of floor.	Depth of scour in front.
11	...	7
12	11	9
13	16	15
14	21	15
15	26	16
16	26	16
17	26	16
18	24½	15
19	20½	14
20	9	12
21	...	8½
31	...	11
32	14½	13
33	14	13
34	17	12
35	...	6½

The general instructions issued by Colonel Brownlow, R.E., Chief Engineer, for remedying the state of affairs at the dam were as follows, but the Executive Engineer, Mr. M. King, was allowed to use his own discretion in working out practical details:—

(i) At a distance of 30 feet above the floor drive a double line of selected piles, 20 feet long, to 2 feet below floor level. In front of the piles put down kunkur and brickbats; fill in the space between the floor and piles with stiff clay puddle to within 4 feet of the floor and work it up thoroughly; on the puddle ram 4 feet of concrete with an upstream surface slope of 1 in 15.

(ii) To fill in the space under the floor, drill vertical holes, 3 inches diameter, at intervals through the floor, and pour in clean dry sand to within 3 feet of the underside of the floor. Level off the top of the sand, then pour in mortar, composed of 1 part kunkar lime and 2 parts coarse river sand, to fill up the remaining space, commencing work from the upper end of the floor in order to force out water through the spaces between the lower line of wells.

(iii) Before the mortar has time to set, drop in small ballast, beginning at the upper end as before, and ram it with heavy crow-bars, to force out any water that may be left through the lower holes, to fill up any bare spaces that may be left in the mortar, and to press the mortar against the underside of the floor.

(iv) A layer of pitching, 4 feet thick, to be laid to a distance of 30 feet below the dam, the top being composed of large heavy blocks the bottom stone or kunkur.

The instructions conveyed are illustrated in Fig. 3, Plate No. 50.

Work was started on the lines laid down, against an estimate for Rs. 21,976, but in the course of carrying out the work considerable additions and alterations were found necessary, which raised the cost of the work to Rs. 27,455.

As the holes scoured out above the dam had got silted up to a depth of about 12 feet, these deposits were not disturbed, but a bed of good clay puddle, 4 feet thick, was laid on them, with $2\frac{1}{2}$ feet of concrete on the top. Upstream of the concrete, a length of 30 feet of dry pitching was laid across the greater part of the nadi to a thickness of 3 feet. Much of the space under the floor had also become silted up; the use of dry sand was, therefore, unnecessary to fill up the lower portion, and the whole of the upper part was completely filled with sand mortar. For this purpose 138 holes were drilled through the floors of 11 of the bays, the number varying from 10 to 13 in each bay.

Considerable changes were made in the original proposals for a talus below the dam of a length of only 30 feet of pitching with heavy blocks on top; intead of this, a talus 88 feet long was put down. Immediately below the lower line of wells a layer of concrete, $2\frac{1}{4}$ feet thick, was placed over 3 feet of clay puddle, with a stepped masonry wall beyond to prevent the clay from being washed away. Below this wall was put 50 feet of pitching, 5 feet in thickness, of which the upper 35 feet was grouted, and below this again more dry pitching, 3 feet thick, the stones at the lower end being laid in mortar for 5 feet to prevent the material above from being washed away.

In putting down puddle and concrete above the dam, considerable difficulty was met with from spring water. This was overcome to some extent by throwing out earthen dams first from piers Nos. 26 and 27, and then from piers Nos. 18 and 19, the work being done in three portions. Another earthen dam across the nadi below the railway bridge sent all the water in the stream down the Hindan cut. Springs were specially troublesome on the right and left flanks, where a number of cast iron pipes had to be placed under the puddle to carry off the spring water. To fill up cavities under and to within 2 feet of the floor, some fine concrete was first laid, made of sand, lime, and small ballast, and stirred as thoroughly as possible with iron rods to cause it to spread quite level. The remaining space was then filled up to the underside of the floor with sand and lime grouting, and carefully rammed with iron rods to fill up all interstices. This practically resulted in laying a second floor below the old one, and fresh holes subsequently drilled through the masonry showed that the two floors had been united into one. There was no difficulty in putting down the talus below the dam.

The quantities of work as executed, the rates, and total cost are shown in the following statement:—

Quantity of work.	Rate. Rs. a. p.	Total cost. Rs.
Earthwork, 1,199,054 cubic feet	4 11 0	4,861
Timbering, 288 " "	3 5 9	968
Sheet piling, 370 lineal "	10 1 8	3,739
Driving piles 370 " "	0 10 7	245
Clay puddle, 81,712 cubic "	14 3 6	1,162
Stone pitching, 68,947 " "	6 6 0	4,398
Concrete, 32,414 " "	18 6 10	5,974
Pumping, 90 days	24 15 7	2,248
Drilling holes in floor, 136 number	1 14 0	262
Mortar under floor, 12,992 cubic feet	11 4 7	1,467
Hire of boats, 560 days	2 0 0	1,120
Grouting, 16,897 cubic feet	5 15 1	1,011
Total		27,455

Preparations for the work were begun in October 1880, but it was not actually started till December, and was completed in June of the following year before the rains set in.

This note has been compiled from official and demi-official correspondence in the file, and from information given by Mr. P. Denehy, Assistant Engineer, who was in charge of the dam when it was damaged and while the repairs were being carried out. At the time the flood occurred and the road and railway banks were breached, Mr. Denehy was of great assistance in procuring boats and passing people from one bank to the other—services which were acknowledged by the Government of India.

PAPER No. 29.

On Automatic Puddling of Channels.

The necessity of preventing loss by percolation in State channels has long exercised the attention of Irrigation engineers.

In the United Provinces we have done very little in the way of puddling perimeters except when the leakage was so great as to scour the neighbouring lands. Where such a course has been adopted, I may say that it has been very successful, but it is expensive, and difficult to carry out in seasons of strong demand. The cost would greatly depend on the length of carriage of the puddling material, but as a general rule the operations would double or treble the capital outlay of a channel. Indeed, Mr. Field, in his note of 29th January 1902, considered that the lining of a 3-foot minor would cost Rs. 5,000 per mile, and would only lead to a return of Rs. 150 a mile. I think the above reasons are sufficient to show why schemes for lining channels by manual labour have not progressed.

In Mr. Kennedy's paper on "Irrigation in the Western States of America" he has dealt with the same subject, and has shown that it would be justifiable to spend Rs. 2,000 to Rs. 4,000 per mile on lining smaller channels. This is, however, not a promising condition, as even when the work was completed, there would be considerable difficulty in preserving its efficiency.

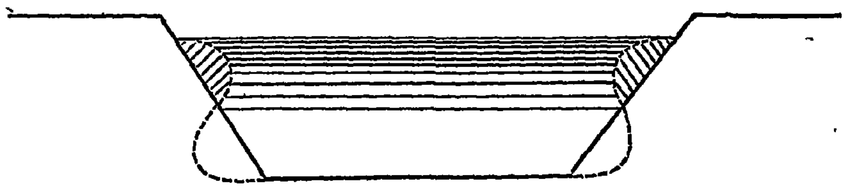
That the question is no doubt a burning one, is vouched for by the words of Mr. Kennedy when he says "Both in America and in India the consensus of opinion is that usually between 50 per cent. and 60 per cent. of the whole canal supply is lost between the canal head and the fields."

Personally I have long felt that as regards channels, the engineer's only practical cure for this serious leakage rests in getting the running water to puddle the perimeters. In Mr. Beresford's paper of 1875 he alludes to this matter and says "I have seen specimens cut from a berm, and from the bed above an expansion fall, that could not be told from the best blue clay of a jhil." I think, therefore, a very fair solution of the question would be established, if we adjusted our channels to such conditions, that the flowing water was continually forming a sedimentary coating on the perimeters. Nor do I think it is necessary that the material should be so visibly clayey, as described by Mr. Beresford. Proofs are forthcoming to show that the gravelly deposits in the pucca channels of the Dun Canal have impermeable properties, and on the Eastern Jumna, the cutting of very sandy looking berms has led to leakage through the slopes of embankments which were previously dry.

The sand of the Jumna near Delhi has apparently no impermeable properties, but when it gets between the loose rubble stones of the Okla weir it certainly holds up water. Similarly, on the Ganges at Hardwar, I have seen old boulder crates so traversed by gravel as to become masses of watertight conglomerate. I think these phenomena are mostly due to the presence of cementing material, which would only be detected by skilled observation.

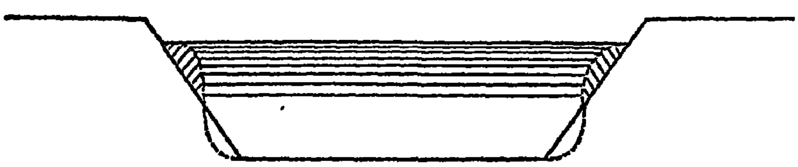
Guided by these facts, I have for several years been remodelling channels so as to obtain the required lining. I think I learnt most from studying distributaries which had been treated by Captain J. O. Ross, R.E. some 22 years ago in the Meerut division. That officer was most zealous and careful in recording his reasons for selecting sections. His favourite slope was $\frac{1 \cdot 01}{5283}$ and this was invariably used after the second mile had been reached, if it were possible. In some cases he was debarred by aqueduct floors, or by some other reason, and then was obliged to use flatter slopes. Where the $\frac{1 \cdot 01}{5283}$ slope was adhered to,

the channel had altered to the following cross-section as indicated by the dotted line:—



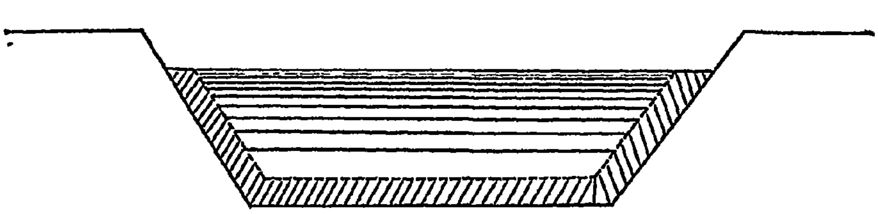
No berm growth had formed except at the level of full supply. At this point ugly projections grew, which fell in during the rains and tore away some of the banks.

Where the slope$=\frac{\cdot 80}{5280}$, the growth was somewhat better, as thus—



There was more formation than erosion, but still there was erosion.

One distributary, however, had the most perfect section, so much so, that I at once asked the subordinate what the gradient was. He replied thoughtlessly that it was $\frac{1\cdot 6}{5280}$. The answer amused the other engineers, as they knew it was contrary to my theories. However, when Captain Ross's book of sections and notes was produced, we found that he actually apologized for being obliged to use $\frac{\cdot 56}{5280}$ on this channel, in order to command a tract lower down. The section was perfectly symmetrical,



and had a lining of 6 to 9 inches throughout. Though the channel was in embankment, with borrow pits in the neighbourhood, they were all quite dry and growing rabi crops. Hence I need hardly state, that from thenceforward I have generally used that slope in remodelling channels, once the second mile was reached. In 1899, when I commenced operations on the Eastern Jumna, where silt had always given trouble, I allow I was somewhat afraid of trying

such a flat gradient, and began with $\frac{\cdot 120}{5280}$ and $\frac{\cdot 80}{5280}$, but subsequent experience has shown that $\frac{\cdot 56}{5280}$ has given excellent results.

At present my belief is, that the slowest working slope is the best. Some officers have done good work with $\frac{\cdot 45}{5280}$, but below that it was found difficult to get the work true enough for water to flow.

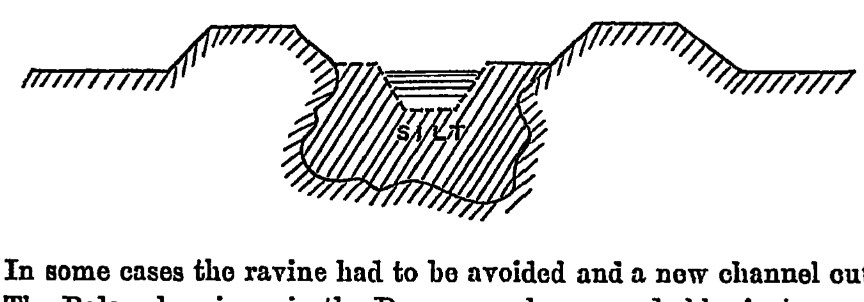
I may here say a word about silt troubles in distributaries. I am quite positive that they are due more to erosion in the parent channel, and elsewhere, than to want of velocity in the branch. If a branch takes off where the main is in a turbulent condition, it will silt no matter what slope is given. The plain remedy for such cases is to look to the regulator, and arrange to secure steady motion in the canal. I have found a distributary at the fifth mile with 2 feet silt in it, although the gradient was $\frac{2\cdot 5}{5280}$. This was due to a fall in the fourth mile having too much draw over it. The channel was scouring so badly, that the water was charged with silt, and the village outlets were useless, as the field channels got choked at once.

As regards results from the regrading of channels on the above principles, I have no difficulty in showing increase of revenue and area on the canals of the United Provinces. There is, however, this fact that the regrading was attended with other improvements, such as—

- Abolition of tatils.
- Greater carrying power of distributaries and change from constant flow to intermittent.
- Reduction of superflous outlets.
- Conversion of lift to flow and generally better command.
- Introduction of electric communications.

Each of these changes should be credited with a share of the progress, but how much it is impossible to say. Remodelling has been effected between Ranipur and Pathri, Ganges Canal, where the breaking up of a rapid slope, and automatic puddling, must have largely brought about the remarkable result, *i.e.*, a supply of 19 cusecs was saved out of a previous total of 42 cusecs. The net outcome of the work was, that an outlay of Rs. 21,000 has produced a capitalized economy of water amounting to Rs. 3,80,000.

Similarly, the regularizing of Fatehpur and Lakhanwala minors, gives very remarkable results. These channels are in the Dun, at the tail of longish systems, and have had no better chance of getting increased volume. They have simply improved on account of the regrading giving better command, and producing watertight channels. Both these minors and those at Pathri had large ravine sections, which have silted up in the most symmetrical manner as per sketch.



In some cases the ravine had to be avoided and a new channel cut.

The Balawala minors in the Dun are also remarkable instances of how gravelly deposits can stop percolation. They are quite new, and when first constructed the water only reached a few furlongs. Once the muddy fluid was admitted, the discharge extended further, and further, and now the tail is reached with the loss of only a small fraction. Its longitudinal section is just the same as those of the Fatehpur and Lakhanwala.

As regards results obtained in the plains, I may say that the Khekra can run 93 cusecs and has a total length of 19 miles. The record figures before and after remodelling stand thus:—

Year.	Aggregate of cusecs.	Running days.	Area.
1896-97	10,787	309	12,365
1901-02	9,434	172	13,432

These are very satisfactory results, as the number of running days was cut down by 56 per cent., total volume by 10 per cent., and the area increased by 10 per cent. Such are hard facts, but still they exclude the inestimable value of the reforms effected. Instead of being mobbed by petitioners about drying up crops, the canal officer now has to investigate only an occasional complaint, and has even been thanked by grateful cultivators for the work that has been done. Over and above all this the country is immensely improved by the reduced number of running days, as this change prevents flooding of village roads, and allows of a freer cartage of produce.

I would draw attention to the great advantage gained in adopting the slow slopes with a series of drops, as it enables the engineer to give command when he wishes, and lower the surface level when it is desirable to do so.

These improvements have been pushed more on the Eastern Jumna and Upper Ganges than elsewhere. They all have produced gratifying results, but though some of the increase is due to automatic puddling, yet a large percentage must be credited to the other factors already quoted.

In this note I have not alluded to the Lower Ganges or Agra Canals, where the slopes of the distributaries have generally ruled low, on account of the conformation of the country. Though the duties on these systems are high, it should also be borne in mind that the first class crops are fewer, and less waterings are given.

In conclusion, I may say that the automatic puddling of the main channels is now being taken in hand, and valuable progress has been made on the Eastern Jumna Canal, the Etawah Branch, and Fatehgarh Branch.

Once the navigation has been abandoned on the Upper and Lower Ganges, the same system can be adopted on those channels. It consists in converting the bridge-floorings into small falls, and berm-cutting the tight places. The main point in this scheme is to remember that the work must proceed upwards, and not downwards from the head of the canal. If the latter course was adopted, the sedimentary lining in the lower reaches would be indefinitely delayed.

Note Proofs of Automatic Puddling, collected by Mr. A. C. Laurie, Executive Engineer.

Approach channel, Baldi aqueduct.—This channel is 700 feet long and in very high embankment nearly all the way, its maximum height being 20 feet above ground-level. It is composed of almost pure gravel and small boulders, with outer slopes at 1 to 1 pitched with boulders. Water first admitted towards end of August 1903, when the loss actually calculated from discharges was 27%. It was run almost the whole of September, but there were few muddy floods during that month, and the diminution of percolation was 4% or 5% only. On 2nd April 1904 a freshet came down, and advantage was taken of it to run a supply through this channel, and the loss immediately fell to 9%. It has run continuously since then, and discharges taken on 6th June showed a loss of only 7%, and one taken on 27th July current year gave 1·8% loss. The process going on is, no doubt, one of conglomeration, of which numerous evidences, in the existence of huge blocks of a sort of natural concrete, are to be found almost everywhere in the Dun torrents.

Balawala and Nathuawala minors.—The first mile of these channels runs in an old excavated channel, which was given up as hopeless some 12 or 15 years ago. The bed and sides consist of boulders, from a cwt. to a ton in weight. The original bed had a very rapid slope, which prevented any deposit of silt; this was flattened by introducing falls at every 100 feet or so.

On first admitting water in June 1903, a discharge of 18 cusecs did not reach beyond the first furlong for three days. The process of natural puddling was all this time going on, and gradually the water reached further and further. By the 25th August 1903 the loss over the whole length of 9 furlongs, with a discharge of 18 cusecs at head, was only 24%. This channel has been run, whenever supply admitted of it, during the rabi, and again since the rains have broken. The loss in the total length of 9 furlongs is now only 9%. In addition to filling up the interstices underground, a deposit of one foot thick has been formed all round the wetted perimeter. The channels were dug one foot wider and deeper to allow for this deposit.

PAPER No. 30.

On the Distribution of Water by measurement.

This subject having been very fully dealt with by the Irrigation Commission in paragraphs 274 to 289 inclusive, of Part I of their Report, it is hardly necessary to do more than reprint these paragraphs here, as more convenient for reference during discussion of the question. Put very briefly, the Commission writes that:—

(1) *Assessment* by volume will be difficult and can only be possible in the distant future.

(2) That *Distribution* by measure is highly advisable and recommend that "strenuous and continuous efforts be made to perfect the system" (paragraph 290).

(3) That when perfected, it would probably take the form of supplying, by measure or module, a fixed discharge for such periods in each crop as should be sufficient to mature the known area cropped on each outlet.

(4) That eventually, but very gradually, when this was perfected, it might be possible and advisable to introduce the lease or block system.

(5) That in any case the mere *distribution* by volume, instead of the present distribution by guess, would lead to great economy (paragraph 286).

In paragraph 285 is described what we require, and how the system would be gradually introduced.

The disadvantages of the present methods and the wastage incurred are not perhaps sufficiently accentuated in the Report, and these may be put down as follows:—

(*a*) The supply to any outlet is not proportional to its real requirements; the levels in distributary and in water-course governing the discharge are varying, and so often, is the area irrigated or to be matured during any given crop.

(*b*) One outlet by the mere deepening of its water-course may be drawing double what was intended, whereas its neighbour, which for various reasons cannot be, or is not, cleaned out, is starved.

(*c*) Often in famine years most of the upper reach water-courses are so cleaned out, and the tail outlets cannot consequently be properly supplied by the full capacity of the distributary.

(*d*) The same thing happens when the gauge-keeper at the head of a distributary permits fluctuations in supply (and this is very frequent, especially at night). The tail outlets are alternately full and mere dribbles.

(*e*) At present we do not know where our water goes, and we cannot say, either as to our distributaries or our water-courses, which is an economical channel, and which is wasteful. Our statistics, such as they are, are quite untrustworthy.

No doubt during dry years, when water reaches its highest value, many cultivators do take greater care of their supply, but the point is that what may be saved by them, may be more than counterbalanced by the waste of their neighbours, who have more than they can well use. Every irrigation officer must have seen in dry years one area or block drying up, and another close by laying on extra waterings to crops not in the least requiring them.

Extract, paragraphs 274 to 289, from the Report of the Indian Irrigation Commission, 1901-03, Part I.—General.

274. *Charges by volume.*—The advantages of charging by volume are sometimes exaggerated, but they are nevertheless so great and unquestionable that

all who are interested in irrigation are hopeful that it may be possible sooner or later to introduce such a system wherever canal irrigation is practised. Before, however, considering the subject further it is necessary to form a clear idea of what is meant by charging by volume. There may be said to be two systems—the charge by meter, and the charge by module.

275. *The "meter" system.*—Many of those who advocate volumetric charges have in their minds a system under which the users of water will be charged according to the number of units of supply (say the foot-acre, or 43,560 cubic feet) actually used, the quantity being measured on a self-recording *meter* at a distance not very far removed from the field to be watered. In the same way Municipalities or Public Companies charge dwellers in towns for water used for domestic purposes, or for gas, or electrical energy. This may be called the "meter system." But for reasons which will presently be explained it is wholly inapplicable to irrigation in India, except in a few special cases in which the water is distributed over a comparatively small and compact area situated in close proximity to the source of supply. A good instance of this is a moderate sized tank of the kind that we have proposed for the Central Provinces, constructed for the storage of water to be issued as required for irrigating the lands in the immediate vicinity, when there is a prolonged break in the rains, or when the rains begin very late or end very early. In such a case there would be no real difficulty in arranging that the sluice should pass out a constant supply (say 8 cusecs), whatever the level of water in the tank above a certain minimum, except that of providing a reliable man to control the regulation. A charge for the use of this full supply, at so much an hour, might then be fixed; and all the occupiers who were entitled to the privilege might be allowed to take it in turn or on application, for whatever periods might be required, on payment of the prescribed rate. As another example may be cited the case of a large steam pump erected on the bank of a river, or other suitable source of supply, for the irrigation of crops for which there might be a fitful demand. The discharge of the pumps could be easily regulated to some constant quantity; and the manager would probably introduce a scale of charges per hour for the use of a full bore supply, the value of which the cultivators would have no difficulty in appreciating after a little experience.

276. In both these cases there is an element of difficulty which deserves notice, as it occurs in a greater or less degree in all systems of charging by volume. The value of a unit of supply delivered at the sluice of the tank or at the pumping station, is very much more when applied to lands in the immediate vicinity than when taken to lands two or three miles away. The cultivator of the latter cannot afford to pay as much per unit of supply as the cultivator of the former, for it will irrigate a much smaller area. When the whole area under protection is not very large, the difficulty would probably be met by fixing a uniform rate of charge with reference to the average length of lead; but when the area is large a system of payment by volume is likely to break down, unless there is a sliding scale of charges, which should vary inversely with the distance of the fields to be irrigated from the point at which the supply is measured.

277. There are probably a few works on which a system of charging by volume of this kind could be introduced without much difficulty. They would offer a valuable means of testing the advantages claimed for a volumetric system of charge; and, if all these advantages were realized, they would be equally valuable as a means of measuring the increase of efficiency which might be expected on other irrigation works if charges were based on the supply used and not on the area irrigated. But the field in which the meter system of charging can be introduced is very limited.

278. *Meter and module systems compared.*—The basis of the meter system of charging for water is that the cultivator pays a certain sum per unit. In a rainy season one unit of water may be sufficient to mature his crop. In a dry season it may require five units; and in the latter case he will pay five times as much as in the former, although the value of the crop obtained may be no greater and will often be less. In the module system the cultivator pays a rate for the volume of water allowed to pass through a sluice, at prescribed intervals of time, in sufficient quantity to mature his crop, the discharge of the sluice

bearing a certain proportion to the area to be irrigated. In a rainy season, if he has taken one watering for his field, he will have to pay the same as he would in a dry season when he may require five waterings. He pays, in fact, at a moderate and uniform rate per unit for the supply which is placed at his disposal, and not, as under the meter system, at a much higher rate for the amount which he actually takes. Under the latter system, he might in a dry season have to pay more than the crop was worth; while in a wet year he might pay only a trifling sum for a single watering, which nevertheless was absolutely necessary to save a valuable crop that he might not have sown at all, but for the knowledge that he could get the water when he wanted it. On this ground alone the module system, with its lower unit rate of charge, is certain to be more acceptable to the cultivator than the meter system. But there is another reason which renders the latter wholly unsuitable for general adoption.

279. On any extensive canal system the principal care and duty of the irrigation officer is to ensure the regular delivery at the extremities of all his distributaries (either simultaneously or according to a fixed system of rotation) of the supply required for the irrigation of the crops or the preparation of the land for sowings. When arrangements have thus been made for sending water over a distance, which may often amount to over a hundred miles, for the use of a particular water-course, it is obvious that it ought to be paid for, whether the owners of the water-course elect to take the whole of it or not. A very large percentage of the supply actually rendered available at the head of the water-course has been unavoidably lost in transit, and when water has passed a certain distance down a distributary, it cannot be sent back again or utilized elsewhere, if those to whom it has been allotted decline to take it. The case is altogether different from those which have been considered in paragraphs 275 and 276, in which the source of supply is close at hand, and can be readily tapped to meet the requirements of a consumer on application to the man in charge.

280. For this reason the meter system is quite inapplicable to large irrigation works, and the module is the only possible system of charging by volume; the function of a module being, not like that of a meter to measure the volume of the supply passing through it, but to *control* it, so that when fully open the discharge will be constant and will not rise above or fall below the rate at which it is set, whatever variations may occur in the level of the water in the supply channel, or in the drawing capacity of the water-course, which can very often be considerably increased at the will or by the action of the owners. The consumer could if he pleased reduce the discharge, or close the module altogether; but he would not ordinarily be entitled to any reduction in the charge made if he failed to utilize the whole supply for which the module was set. We are convinced that this is the only system of charging by volume suitable for India, except in the special cases referred to already; and it may be added that in all countries in which an attempt has been made to charge by volume—in Italy, in Spain, and in America—it is the module and not the meter system which has been adopted.

281. *Difficulties in obtaining a module.*—It is often supposed that the main objection to the module system is the difficulty of obtaining a satisfactory module. It is of course obvious that it is impossible to enter into an engagement with each individual cultivator, or to erect a module on every holding. On the other hand, charges by volume cannot be recovered from a larger unit than the village; for there would be great and insuperable difficulties in distributing either the charge or the supply between the members of two or more villages. But a village is often irrigated for one or both sides of two or more independent water-courses, according to the configuration of the ground, so that the unit for the purpose of volumetric charges must always be the water-course, and each water-course must be supplied with its own module. There is another reason which limits the size of the area which may be supplied from one module, to which allusion has already been made in paragraph 276—a uniform rate cannot be charged per unit of supply, when there is very great variation in the lengths of the channel leading to the consumers' lands.

282. The construction of a module in itself presents no great difficulty, if reasonable amount of attention can be given to its working, and too great a

degree of accuracy is not insisted on. Mr. Kennedy, Superintending Engineer in the Punjab Irrigation Branch, who has recently studied the American system of irrigation on the spot, has told us that the system of charging by module is in general use there, and that the modules, although not very accurate, give satisfaction. But they are applied to channels of much greater capacity than that of the ordinary Indian village water-course, which ordinarily varies from half a cusec to four cusecs; and the American farmers are much better able to look after themselves and each other, and to satisfy themselves as to the fairness of the distribution and the correct working of the modules, than the Indian rayat. In Northern India there would probably be on an average five modules in every irrigating village, scattered all over the country, and under very little supervision. They would have to work with a very small and often a hardly appreciable head; their working must not be affected by silt; and they must be so designed that they cannot easily be tampered with. Clockwork and similar complications must be regarded as out of the question. All these difficulties are considerable, but we are not prepared to say that they cannot be overcome. If it were once decided to take up the question of volumetric distribution in earnest, we have no doubt that the ingenuity of the irrigation officers and of other inventors would be equal to the occasion; and that a satisfactory module could be devised, if all conditions and requirements were clearly understood. The fact is that no serious attempt has hitherto been made to introduce modules, because it has no doubt been felt that the want of a good module is one of the least of the difficulties in the way of a volumetric system of assessment.

283. *Objections by cultivators.*—The first of these difficulties is the distrust with which any change in the present system would be regarded by the Indian cultivator, who knows nothing about the quantitative determination of the supply of flowing water, but has been accustomed from time immemorial to pay a revenue dependent on the area and nature of the crops which he cultivates. A water-rate based on these factors is intelligible to him; a contract for a given supply of water is not. He does not know how far the supply for which he contracts may be curtailed or tampered with by the petty officials who must be employed even under a system of volumetric charges, or how he is to bring complaints of short supply to the notice of the canal officers, and to establish them. Then there is the question of the internal distribution already alluded to in paragraph 271. The cultivator may find himself called on to pay a larger share of the sum charged on the water-course than is represented by the volume that he has actually received, for he cannot always hold his own against his neighbours. Under the present system, however much he may be deprived of his fair share of the village supply, he is only called on to pay according to the area which he has actually irrigated, and he can claim remissions on that if the crop is below a certain standard. He can understand the principle of "no crop, no charge," which is now followed as far as possible in canal administration; but he will have no confidence in a system in which his liability for water-rate will be wholly independent of the area and quality of his crop.

284. *Difficulty in devising a scale of charges.*—The canal officer will have to meet another difficulty, which is presented by the problem—"given all details of the supply, the rainfall, the irrigated area, and the revenue from water-rates during a series of years, to propose a scale of rates of charge per unit of volume which shall secure Government against loss of revenue, and shall be acceptable to the cultivator." The problem has only to be stated for the inherent difficulties to become apparent. In practice, no doubt, it would be necessary, as in the case of the Eastern Jumna Canal experiment, to reconcile the cultivator to the new order of things by proposing charges which involve a considerable loss of revenue to Government—a loss which would be continuous until the scale of charges could be revised; and revisions could, for obvious reasons, only take place at long intervals.

285. *Distribution by module.*—For these reasons we think that, even if a perfect village module could be brought out to-morrow, the day on which a volumetric system of charges could be introduced into India would be distant.

But we do not say that it will never come if systematic efforts are made to prepare the way for it. The first step to be taken is to establish in a selected canal, or section of a canal, *a volumetric system of distribution*, the present system of assessment remaining unaltered. Mr. Kennedy, who has devoted a great deal of attention to this subject, has designed and constructed a module for the head of a distributary, which can by a simple adjustment of weights be set to pass any prescribed supply, within certain limits, into the distributary, whatever the variation in the canal supply or in the drawing power of the distributary. He says that this module has now been under trial for two or three years, and that it may be relied on to pass the particular supply for which it is set within 2 per cent. We have no independent evidence of the practical efficiency of this particular module; but, however this may be, we think that there should be no difficulty in controlling the supply entering a distributary to within 5 per cent. of whatever may be prescribed. When the supply entering the head of a distributary can be controlled within these limits, the first great step has been taken to secure regularity in distribution. The next step will be to construct suitable modules at the heads of all the branches and minors taking off from the distributary, so that each shall receive a share proportioned to its requirements. The third step would be to construct the modules at the heads of all water-courses, the volume to be charged for depending on the lengths of the periods during which the head of the distributary was open. The periods of flow and closure would be regulated in accordance with a prescribed programme, and would be well-known to every cultivator. The present system of assessment should not be changed; but after a few seasons of regular and systematic working, the cultivators would understand what was meant by an hour's working of their modules at full bore. After a time they might be allowed the option of contracting for the season or for a number of seasons; the terms of the contract being a full bore for so many days in the week or other suitable period, the length of the time being subject to rateable reduction when the supply in the canal was insufficient; but each reduction to carry a proportionate reduction in the amount to be paid. If the system became popular, the change would in course of time be made voluntarily, or it might be made compulsory when it was found that the majority of the cultivators preferred it.

286. If satisfactory modules can be devised, it appears to us that it is only in this way that the present system of assessment by area can be changed for one of charge by volume. It is only by proceeding gradually, and enlisting public opinion in its favour, that such a change can ever be introduced in India. It is probable, however, that distribution by modules in the way proposed would result in great economy, even if the people preferred to adhere to the present system of assessment. The more systematic the distribution, and the greater the certainty of the cultivator as to the supply he will receive, the greater will be the efficiency of the canal, whatever system of assessment be adopted. The attention which has been paid of late years to the method of working distributaries, and especially to the proportioning of outlets to requirements, has been followed by remarkable improvements in the duty and expansion of revenue. But still better results will be obtained when the distribution is regulated by more accurate measurements, such as are now recognized as essential to real progress in every department of practical science, and in a great many industrial undertakings.

287. *Modules on Sone Canals.*—We have already pointed out that there is no effective inducement to the peasant on the Sone Canals to economize water; as he cannot utilize and portion off the supply at his disposal on lands outside the boundaries of his block. These boundaries cannot be extended or removed at present, because the supplies to which these blocks are entitled have never been quantitatively defined. Until this is done a true system of charging by volume cannot be introduced. If, however, modules were introduced on these canals, and the quantities to be assigned to each block at different periods of the season were definitely fixed, it should be possible after a season during which the cultivators would have learned to understand the new system, to grant future leases in terms of the volume to be supplied, and to fix no boundaries to the area to which water might be supplied.

288. *Application of the method of volumetric charges to other provinces.*—A system of long leases based on charges by volume will be very suitable for some of the works in the Bombay Deccan, as soon as sufficient experience has been gained in the working of modules to enable the cultivators to understand to what quantities of water they are entitled. The substitution of charges by volume for the present system of a consolidated wet assessment, such as obtains in Madras, would involve such a change in the whole system of land revenue assessment that we cannot recommend it, at any rate on existing works. We doubt also whether a system of volumetric charges would ever be suitable for Sind, where the supplies to the canals are subject to great fluctuations.

289. *General conclusion.*—Our general conclusion on this question is that the module system of charging by volume cannot, in spite of all its advantages, be safely introduced in India, until a system of distribution by modules of the type which it may be proposed to use has been in force for a time sufficiently long to enable the people to understand what is proposed; and that even then the change in the system of assessment should not be forced, but be introduced gradually as the people learn to appreciate its advantages. It is, however, an end to be aimed at; and irrigation officers should be encouraged to design and experiment on modules which will be suited to the conditions to be met with in practice, until the work of distribution can be carried out with all the regularity and certainty which is essential to the success of any scheme of charging by volume.

PAPER No. 31.

Distribution of Stored Water in the Deccan.

The distribution of water from the Government canals in the Deccan is, in the author's opinion, most unsatisfactory; the rules under the Bombay Irrigation Act prevent the cultivators from obtaining water without troublesome application and formalities. It is argued that as there is generally rain in the Deccan and monsoon crops can be grown without irrigation, and also that as the water has to be stored, the distribution cannot be made upon the same lines as in Sind and the Punjab, where irrigation is a necessity.

Again, it is argued that during famine years the water will be wanted for monsoon and rabi crops over a very large area and so the main canals and branches are constructed to command a much larger area of land than they can ever supply and canals in the Deccan have many cross drainage works and are expensive, but, on the other hand, no system of field distribution is provided and as the field channels may have to pass through lands not in a cultivator's own village it is often difficult for an individual to obtain the channel he wants.

The author maintains that the broad principles for successful irrigation are the same for the Deccan as anywhere else and that they are :—

(1) The delivery of water to every field to be irrigated.

(2) An assured supply of water available at known times.

(3) Freedom to cultivate any crops with the supply available.

For all ghat-fed storage works in the Deccan there is an assured supply of water; the crops which require water in addition to rainfall are 8 months' crops from mid-June to mid-February, rabi crops from November to February, and perennial crops for the whole of the year.

The minimum area of these crops which can be irrigated from any ghat irrigation work is easily calculated, and the system of distribution should be laid out to irrigate that area and as there is always plenty of land the best land should be selected; if the canal and distribution be made for area three times as large as it is calculated possible to irrigate it will be amply large enough.

For the proper proportion of 8 months' rabi, and perennial crops the Nasik and Khandesh Bhandaras give reliable facts, the land irrigated under them is divided into a quadrennial system, or a triennial system and the latter is the favourite. In the triennial system about equal parts of 8 months, rabi, and perennial crops are irrigated. The canals system should therefore be laid out to irrigate equal parts of these crops.

It is known that 120 acres of either 8 months of rabi crops can be irrigated by 1 cubic foot per second and it is believed that 60 acres of perennial crops can be irrigated by 1 cubic second.

The quantities of water required to irrigate equal area as will then be in proportion to—

$$\frac{8}{120} \text{ to } \frac{4}{120} \text{ to } \frac{12}{60}$$
$$= 16 \text{ to } 8 \text{ to } 24$$
$$= 2 \text{ to } 1 \text{ to } 6$$

Hence neglecting losses—

$\frac{6}{9}$ must be reserved for perennial.
$\frac{2}{9}$ for eight-months.
$\frac{1}{9}$ for rabi.

When the system of the distribution has been designed and laid out, then the water must be run through the channels whether the water is used for irrigation or not. There is no doubt that all the perennial water will be used up when the cultivators are satisfied that the supply is certain, and the quicker they can be convinced of this the quicker will the water be used.

Similarly, all rabi and eight months' water will be used up when confidence is gained.

At first applications will be received from lands beyond the designed system asking for the water which is being run to waste, but these must be rigidly refused, for if the water allotted to any village be taken away from it, the villagers will at once lose confidence in the supply.

No mention has been made of monsoon irrigation in the preceding calculations: it is a very uncertain quantity and only taken when rain fails. The canal, however, has to be made big enough to discharge the total of the three seasons, 8 months, rabi, and perennial, and as rabi, water is not wanted during the monsoon the canal could discharge the full supply and irrigate when necessary an area of monsoon crops at least equal to the rabi area. As the system of distribution is to be made three times the irrigable area, there is plenty of space for such irrigation without extension of the canal beyond ordinary requirements. The discharge of the canal for 8 months of the year will thus be double the discharge for the hot weather.

In regulating the supply, if the size of the outlets be fixed to give the discharge for the 8 months, then for the hot weather seasons the supply should be given for half the time only. Thus the outlets would run constantly for 8 months and for 3½ days a week during the hot weather; the distribution of water can then always be made at the heads of distributaries which should be run full whilst they are working. Regulators should be provided in the main canal if necessary to get the full depth into the distributary with half supply in the main canal.

In years when there is good rain during the rabi season, and rabi water is not issued in the designed quantity, there will be more water in the tank during the hot weather than is required for the perennial area fixed. This water may be used for special hot weather crops which are much valued in the Deccan, but the cultivators should not be allowed to increase their perennial area above the fixed quantity.

To sum up. For successful irrigation from a ghat-fed tank the whole of the water must be issued every year from the tank and distributed in fixed proportions to the villages commanded by the canal.

PAPER No. 32.

Deccan Irrigation could be made to pay.

The loss of water by evaporation from a tank 250 feet deep is proportionately much less than on one 50 feet deep and at Bhandardarra 8,000 million will be available for issue from the tank; the monsoon-flow would provide sufficient for the canal for 4 months of the year and the 8,000 million will provide 380 cubic feet a second for the remaining 8 months. The headworks of the canal are 50 miles down the river and as there is a small stream flowing for most of the year, a deduction of 50 cubic seconds or 1 cubit second per mile should be ample provision for loss on the way to the headworks. The canal on the left bank reaches the ridge in about 45 miles and a deduction of 30 cubic seconds will be sufficient for loss in the canal; there are thus 300 cubic seconds available at the heads of distributaries.

If the areas of perennial, rabi, and 8 months' crops be made equal, then as during the monsoon there is enough water in the river the water available would be—

4 monsoon months from river			
4 rabi months	. $\frac{1}{3}$ for 8 months = 100 cub. sec.		
	$\frac{1}{3}$ for rabi = 100 „ „		400
	$\frac{2}{3}$ for perennial = 200 „ „		
4 hot weather	. $\frac{2}{3}$ for „ = 200 „ „		200

Note—(400 cubic seconds for 4 months = 1,600
200 × for 4 „ = 800

Total . 2,400=300 × 8 months.

Taking a duty of 120 for the 8 months and rabi and 60 for perennial the areas irrigated would be—

8 months	120×100=12,000
Rabi	120×100=12,000
Perennial	60×200=12,000
	36,000

To this may be added 12,000 monsoon crops which would be irrigated without drawing on the storage, so that the total area which could be irrigated=48,000 acres.

It has been shown in the note on Nasik Bandhara irrigation that a rate of $9\frac{1}{4}$ rupees per acre should eventually be obtained for this area =Rs. 4,44,000, but that to allow for good rains when irrigation is not wanted a deduction of about one-fifth must be made so that the average revenue would be Rs. 3,55,200.

It should not cost Rs. 55,200 to work such a system, so a net revenue of Rs. 3 lakhs should be derived and this is equal to 4 per cent. on 75 lakhs.

Other tanks in addition to Bhandardarra can be constructed in this Pravara River Valley and another 8,000 million of water are available and can be stored and it is possible to irrigate 100,000 acres of land making it more valuable than the best garden lands under the Nasik Bandharas.

The Superintending Engineer on special duty in the Deccan is surveying the ghat rivers and working up similar schemes.

The author is convinced that these ghat-fed tanks and canals can be made to pay if they are constructed on a large scale, the water properly distributed, and the rates for irrigation raised, and he also thinks that some colonization scheme should be devised. The immediate effect of such a canal is to raise the value of the lands irrigable by Rs. 100 an acre and the Bhandardarra scheme alone would add 50 lakhs to the value of the land, and with additional

tanks 100 lakhs. At present none of this increased value is received by Government. The existing owners are not able to immediately cultivate all these lands with the higher class of farming required for irrigated crops and the development of the canals is delayed.

The author thinks it practicable and fair to the people, and for Government to acquire the whole land irrigable, to construct the water-courses and to redistribute the land at an increased value, giving back to the original owners as much land as they can cultivate with irrigation, so that they will derive profits more than they had before, and selling the remainder. Some such system of colonization is necessary.

PAPER No. 33.

Small Tanks in the Southern Mahratta Country.

In the south of the Bombay Presidency are a number of small tanks used mainly for rice irrigation; they are situated in an area where the south-west monsoon rain is regular and certain, but comparatively light, varying from an average of 50 inches on the western border to 25 inches on the eastern border of the area; the rainfall is sufficient for ordinary crops, but not persistent enough for rice, and the tanks are used partly to divert the water from streams on to the rice fields and partly to store water for irrigation during breaks in the rains. There are more than 2,700 of such tanks irrigating over 50 acre each; the great majority, over 1,800, are in the Dharwar district and there are 500 in the adjacent portion of Kanara and 414 in the south-west of Belgaum.

They are shallow, the majority probably having less than six feet of water when full, and the maximum depth is 20 feet. I have not accurate figures of the irrigation, but 473 of these tanks in the Dharwar district irrigate 65,929 acres and have an irrigation revenue of Rs. 1,73,909 an average of 2·64 rupees per acre for monsoon irrigation.

The great majority of the tanks are for monsoon irrigation only and have not water for rabi crops, but one of the biggest at Haveri irrigates 331 acres of garden land and 141 acres of rice land. The consolidated revenue including land and irrigation revenue for the tank is Rs. 4,029 for the garden land=12·2 rupees per acre and Rs. 1,032 for rice=7·5 rupees per acre. The rates on these tanks show the value of monsoon irrigation.

PAPER No. 34.

The Nasik Bandhara Irrigation.

In the western portion of the Nasik district, where the streams are fed by the south-west monsoon and flow to the east through a district with only 25 inches of rainfall, the waters are diverted from the streams and used for irrigation. The streams have rocky beds and a steep fall of some 40 feet in a mile and rich soil at places on the fields by their side.

Bandhara means weir.

A typical bandhara is a low masonry weir built just above a rocky fall in the stream, with a channel taken off usually on one side only; a masonry regulating wall is built up to above flood-level in continuation of the weir, and a hole about 2 feet square provided as an outlet to pass water into the canal.

The channel is taken along the bank of the stream with a bed-fall of about 1 in 2,000 and rapidly gains command of the fields a short distance below weir.

The outlet in the regulator wall is left open; planks or needles, if provided, are neglected and not used and are lost by the cultivators; and so during floods more water passes through the outlet than the channel can discharge, and escapes are provided for the excess water. When the weir is on good rock the side of the channel immediately below the outlet, is formed by a masonry lining wall with top at full supply level to act as an escape, and a little further down the channel a second masonry regulator is built to check the excess flow of water down the channel.

The typical bandhara is, therefore, a masonry weir above a rocky-fall, a regulator wall and outlet, and masonry lining wall to the side of the canal immediately below the outlet, a second regulator across the channel below the lining wall, and a channel leading to the fields to be irrigated. When the rocky-fall is of sufficient height the channel is at once above ordinary flood level and not liable to be submerged in parts and breached by floods.

Such favourable conditions, though not at all unusual, are, of course, not universal and every variety of conditions is to be met, from small walls 6 inches high to big massive weirs 20 feet high and 10 feet thick ; channels which are at once above flood-level and channels which get submerged in low floods and breach their banks constantly unless ample escapes are provided ; channels which command the land to be irrigated without any trouble from cross drainage, and channels which cross numerous small streams or drainage channels and get breached in rain-storms or which cross a second large stream requiring another weir.

The foundations of the weirs also vary from a site with good rock on bed and flanks, to sites with no rock on the flanks and where big floods outflank and cut round the weir and require flank walls and flank embankments to keep the floods in place.

The areas irrigated vary from 5 acres up to as much as 500 acres. The supply of water varies from an occasional uncertain supply from showers which is used to supplement irrigation from wells; to an excessive supply during the monsoon months, a good supply during the rabi months or up to the end of February, and thus a small supply from springs in the river-bed used for perennial crops.

All the best bandharas have a spring in the river-bed just above them and on many of the rivers there is a succession of such springs and bandharas; one bandhara diverts all the water in the river during the dry months ; but a short distance below another bandhara often has just as good a supply as the upper one.

In this system of irrigation almost every engineering irrigation difficulty or problem is to be met in miniature; Government maintain the weirs and masonry works and the cultivators maintain the channels, but when a channel

is being constantly badly breached by cross drainage or flood-water, the question arises whether Government should build a masonry work or the cultivators do the repairs. It is generally easy enough to design something comparatively magnificent, which would prevent the damage, but it is not by any means easy to keep the cost down to a sum which the revenue derived will justify the engineer in expending, and trained experience and skill is required for the maintenance and repairs of these works.

The number of bandharas in the Nasik and Khandesh Irrigation Districts is 361, the area irrigated is 38,976 acres, and the irrigation revenue derived is Rs. 2,27,604. The land revenue is in addition to this amount. The works were mostly constructed before the British Government took charge of them and the irrigation is, therefore, old and long established. The rates paid vary with the supply of water, but the best supply is not equal to that available from a canal, fed from a tank or a ghat river like the existing Nira Canal or the proposed Bhandardarra Tank and canals below it. The rates on the best of these bandharas are, therefore, a good guide to the returns which may finally be expected from a canal and reservoir with a certain supply of water.

The rates paid for irrigation under the Nasik and Khandesh Bandharas where a good supply of water is available are much higher than the average rates under Government canals, moreover they include the risks of bad seasons, whilst on the Government canals water-rates are only paid where a field is irrigated.

For rabi crops, in the Deccan, when the cold weather rains are good and prolonged, irrigation is not required. Such good rains occur about once in five years. Under the Government canals the cultivators have no risks of short supply and avoid paying rates when there is good rainfall, and the canal has to take the loss although the cultivator has a good crop. On the canals, therefore, the rates should be one-fifth higher than rates for the bandharas.

The crops grown under these bandharas, are monsoon, 8 months, rabi and perennial and the land is cultivated in rotation. The area of perennial irrigation is limited by the supply from the springs and is seldom as much as one-third even under the best bandharas. The rate paid is, however, the same for the whole area, an average rate having been settled for each bandhara. So that the rates per acre for the best bandharas are not the rates for perennial irrigation but the average for the four seasons or three seasons, some systems being quadrennial and some triennial.

It has been shown that the water required for rabi, 8 months, and perennial crops, is in the proportion of 1 : 2 and 6; but for 8 months and perennial the first four months irrigation is during the monsoon, when stored water is not required; if Re. 1 be taken from the monsoon period Rs. 5 for rabi, then the rates would be :—

Monsoon	Re. 1
Rabi	Rs. 5
Eight months	" 6
Perennial	" 26
	4) 38
	9

or Rs. $9\frac{1}{2}$ for the average rate; if, as explained above, one-fifth be deducted to get a rate to correspond with the rates of the bandharas then the rate would be $8\frac{1}{4}$ rupees; and this is not excessive.

Attached is a list of 47 bandharas irrigating 8,566 acres, for which the irrigation share of the rate is over Rs. 9 per acre; and 10 of them have a rate of over Rs. 12 per acre. If, therefore, the land under ghat-fed tank canals in the Deccan be laid out in the same manner as these Nasik Bandhara lands, on a quadrennial system, for monsoon, rabi, 8 months, and perennial crops, then a return of over Rs. 9 per acre should be obtained eventually for the areas that can be effectively irrigated.

List of Bhandaras with rates over Rupees nine per acre for the Irrigation share.

Rates.	Taluka.	No.	Area assessed.	Irrigation share.	Rate.
Over 12	Pimpalner	47	190	2293	12·20
		48	132	3465	12·37
		49	222	2739	12·34
		51	312	3830	12·28
		52	7	85	12·14
	Kalvan	5	19	231	12·16
		8	18	219	12·17
	Malegaon	43 (Patna)	602	6641	12·41
	Baglan	58	12	157	13·08
		60	39	480	12·31
11—12	Pimpalner	41	382	4270	11·18
		55	154	1761	11·59
	Malegaon	43 (Dabhadi)	380	4448	11·77
	Bagnal	61 (Sampur)	199	2383	11·97
		72	92	1049	11·40
10—11	Amalner	6	193	2009	10·41
		7	104	1050	10·09
	Pimpalner	39	185	1830	10·40
		50	23	217	10·85
	Kalvan	17	506	5102	10·08
		18	24	256	10·67
		22 (Sadki)	55	563	10·05
	Bagnal	57	281	3042	10·83
		59	291	3022	10·38
		61 (Teharabad)	14	152	10·86
		63	44	457	10·39
		68	422	4541	10·76
	Malegaon	74	204	2084	10·17
9—10	Pimpalner	19	5	49	9·80
		36	120	801	9·31
		38	177	1185	9·05
		53	43	428	9·95
	Dhulia	59	280	2743	9·79
		60	148	1350	9·12
	Kalvan	1	191	1712	9·90
		13	76	695	9·14
	Bagnal	31	158	1509	9·55
		33	635	6113	9·63
		34	325	3132	9·64
		35	108	1034	9·57
		42	36	347	9·64
		64	303	2971	9·81
		66	180	1730	9·61
		67	17	169	9·94
		71	424	3913	9·23
	Malegaon	73 (Walwada)	152	1484	9·56
	Niphad	6 (Bandhara at Wadali)	82	802	9·78

Note on above papers.

The author's object has been to show that Deccan irrigation from the Ghat rivers with storage tanks can be made to pay.

The projects will be protective works and cannot yield returns like the canals from the perennial rivers in the plains, but their value as protective works is so great that they should be constructed wherever possible and if they can be made to pay Government the cost of constructing them, then the money for the works will be readily obtained.

He considers also that the rates for irrigation on the big productive works of the Punjab, Sind, and elsewhere are very low and might be increased and extra revenue obtained for use in constructing works in less favoured places.

PAPER No. 35.

On Remodelling of Irrigation Distributaries.

The most important of the Punjab reports and notes have already been reprinted in Punjab Irrigation Branch* Paper No. 10, and as these, especially the instructions issued for the Western Jumna Canal, contain the fullest details, this need only be a brief resumé of the main lines on which such schemes have recently been worked out: in fact, this may be considered as a preface to Paper No. 10.

As at present worked, our irrigating machine is perhaps the most inefficient device ever devised for doing any useful work. Of the whole supply we take out of the river we lose 40 or 50 per cent. on the way to the fields; and another 15 to 25 per cent. is wasted in application; so that, speaking roughly, the "efficiency" is only from 30 to 40 per cent. The first of these losses we can only remedy to a slight extent by the measures detailed below, and one main cause of the latter loss lies in the fact that the supply which a cultivator has control of is seldom proportional to his real requirements, as gauged by the land he has under irrigation. We do not know where our water goes to within 10 or 20 per cent. in the distributaries and within 30 or 40 per cent. in the water-courses. Our remodelling therefore at present can only aim at improving minor defects, perhaps resulting in an improved efficiency up to 40 or 45 per cent. at the most.

On few canals is the supply sufficient all the year round to fill all channels; on some it is so short that often for months at a time distributaries only run 7 or 10 days in the month. Thus for many years now, rotational supply of one branch after another has been in force, the capacity of each being such as to be able to supply all its distributaries at once, without internal "tatils." This of course saves a great deal of absorption loss, the simplest criterion for estimating which is simply the total lengths of all channels open at one time; and besides it gives each outlet full supply, and enables a given area to be covered quickly and therefore economically.

The time a distributary will be open during a crop will therefore vary on each canal, and therefore the area which each cusec of its *full supply discharge* can irrigate per year will also vary: this figure has been called the "Full Supply Factor," and is the first thing to settle. On the Western Jumna Canal it has been taken at 150 acres; on the Bari Doab Canal it will be higher, probably 170 or 180 acres, since the supply is greater in proportion to the canal capacity. This figure must not be confused, with "Duty," this being the number of acres (per crop or per year) which an average supply of one cusec running full time will irrigate; it varies from 250 to 300 acres per annum. Formerly in designing distributaries it was the custom to assume some arbitrary " Duty" figure, the result being that as soon as the scheme was completed and in full work, it had to be remodelled to a larger section. Of course all distributaries must be calculated to take this full supply, as deduced from the area to be irrigated per year and the full supply factor. The figures just given are those as measured at distributary heads; at canal heads they will be naturally less.

Twenty years ago distributaries could, as a rule, only carry enough for one-half or one-third of their outlets, necessitating "tatils" by sections, endless trouble, theft, waste, and starvation to those near the tail. Now, of course, each can carry sufficient for the whole of its outlets, but it is often rather difficult to know exactly how much outlet orifice area in all to allow for the required discharge, as the average outflow per square foot of outlet water-way will vary a good deal with the command. The following figures, the results of experience, may be useful; they include all absorption losses on the distributary.

* Not reproduced.

For a small minor with poor command the *average* discharge per square foot of outlet water-way may be as low as 3·5 cusecs, or even lower.

For a distributary with fairly good command, a common average value is 4·0 cusecs.

For a distributary with very good command, 4·5 cusecs.

For excessive command 5·0 cusecs.

These figures are for the whole length; for individual cases of course it may be anything. The average area of water-way of remodelled outlets is about 0·50 square foot each, or say usually 2·0 cusecs discharge, and usually the maximum is about double this.

One of these figures would have to be assumed, thus fixing the *total* outlet water-ways on each channel; but in fixing *each* outlet, local considerations of level and previous known duty must be considered. It is more a question of trial and error than anything else, since it is quite useless trying to measure each outlet's "head." This is much too variable, even in the same outlet, to give any satisfaction, and it means a very great addition to the work to be done.

Every distributary should have profile side walls, giving final bed-width and height of full supply every 2,000 feet or so, so that it will be at once apparent, from the height the water flows at, with full supply, whether the channel requires clearance or not. The grading for the first 4 or 5 miles should give critical velocity V_0; below that, as the sand silt is carried out into the water-courses 0·90 V_0, and 0·80 V_0 may be designed. There is no advantage in having V_0 for the whole length, rather the reverse, as with very poor soil scouring might result, and a lower velocity than V_0 will allow the finer sediment to settle on bed and sides, and so give a more watertight channel. If possible one-foot fall at least should be given from the feeding channel into the distributary head, to avoid trouble with low supplies in the former.

Banks should be when first made not less than 1·5 feet above full supply. Whenever possible, each minor or distributary should end in a cluster of outlets, so as never to have a Government channel carrying less than about 4 cusecs. If necessary, the last mile or so of the minor below the cluster may be handed over to the villagers to maintain, for it has been found almost impossible by Government agency to keep very small channels in order—they silt up so quickly that their clearance is best left in the hands of those most interested. There is not necessarily any more loss on a ragged channel than on a well-kept minor, provided the banks are strong. Of course the distributary head discharge should be kept quite steady, otherwise the tail outlets suffer, alternately open and closed; this, however, is at present very difficult to ensure.

The outlets should have cast iron orifices to prevent interference, and be built high and narrow without arching, so that any adjustment in waterway can be effected by simply raising or lowering the flat bricks on the top. They should all be at bed-level, and the ones near the tail should be in the form of open notches so as to take off any surplus or floods. As far as possible, there should not be more than one village on any one outlet, though sometimes this will be unavoidable.

Long water-courses should have large areas on them, or, in other words, the size of the outlet should increase with the length the water has to travel, otherwise we might have little or no water reaching the fields. The usual maximum length of water-course is about two miles.

The main thing is to have no more open at one time than absolutely necessary, and for this purpose usually we have to amalgamate 2 or 3 water-courses into one; thus saving absorption loss at the rate of from 0·20 to 0·50 cusecs per mile, according to the soil.

This brings us to "*Kiaris*" the want of which means often immense waste. Thus, taking the case where the rate of absorption on the field is 0·06 feet in depth per hour, and discharge entering, one cusec, we find that if the size of "*Kiari*" or compartment is 10,000 square feet we will have to put an average depth of water on this area of 0·295 feet before it can reach the

far end. If, however, the compartment or field area was 50,000 square feet, then we would have to lay on an average depth of 0·58 feet before we could cover it all. Such cases are very common and the remedy is hard to find. The size of compartment prescribed in the Punjab is now 70′ × 70′, and this is not at all too large for good soil, though for sandy soil it should be less.

Instructions for Grading and Designing Irrigation Channels, by R. G. Kennedy, Esq., Chief Engineer, Irrigation Works, Punjab, dated 25th August 1904.

It is constantly noticed that the grading proposed for new or remodelled channels is very defective—there seem to be no fixed principles, and each man usually grades according to his fancy; as often as not merely to suit the ground-contours. This is, of course, utterly wrong; all grading should be quite independent of the ground-slopes, and should be such as to ensure a future *régime* of flow being reached at the earliest date and maintainable at the least cost.

2. Now, it is known that in most of our perennial canals, when the supply in the river is at certain times of the year fully silt-charged with fairly coarse sand, that to avoid bed-silt accumulating and blocking the full flow, we must have certain mean velocities called V_0 for certain depths. These figures are given in the preface to the Discharge and Silt Diagrams now in use in the Punjab, and are not mere theoretical ones, but are based on experience and experiment. All main lines and branches, therefore, must have their mean velocities not less than V_0, otherwise there will be trouble; it may be *slightly* more than this, but should not be less. Also the first four or five miles of each distributary, or minor taking out from the canal direct, must have V_0, but below this length provided there are sufficient outlets in the head reach, built at bed-level, to draw off a portion of the bed-sand, it is possible and advisable to grade for less than V_0, gradually decreasing this figure down to a minimum of about 0·75 V_0 at the tail. This will have the further advantage of settling some of the finer mud on the bed and sides of the lower reaches, and so partially staunching the section against percolation. The value of V_0 should, therefore, decrease from the head downwards of a distributary.

3. In some cases it is found that from erosion going on in the branch or feeder, due to too steep grading, the supply entering the distributary is more than just fully silt-charged, and then, to avoid trouble in the distributary head reach, it would be necessary to grade for more than V_0. This, however, will generally be only temporary, and need not usually be provided for. On the other hand, there are cases where so much as V_0 will never be required. Thus, in most of the Inundation Canals where the sand carried in the river is of finer grade than near the foot of the hills a much less value will suffice; how much less is as yet uncertain.

4. Any minor taking out from well down a distributary, for similar reasons, need not be graded so steep as to give V_0, as there, usually, the supply will not be, fully charged; and if V_0 were given, in some cases, where the soil is poor, there would be erosion even with this velocity, simply because the water would be able to pick up enough sediment to fully charge itself.

5. All the necessary data should be shown on the longitudinal section in columns provided for the purpose on the special section paper in use. No cross-sections of channel are required.

6. The proportions of bed to depth will of course vary with the size of the channel. For small minors of ten cusecs the *maximum* limit may be 3·5, for a distributary of 25 cusecs, 4·0, for one of 100 cusecs, 4·5, and for 200 cusecs, 5·0, for 500 cusecs, 6·0, for 1,000 cusecs, 6·0, and so on; these figures being of course rather indefinite.

7. Wherever the water-surface is high above ground-level, in order to avoid all troubles by breaches, the banks should be set back so as to silt up inner berms. The amount set back will of course increase with the danger of bursting banks, *i.e.*, with the height of water above natural surface, and be

inversely as the strength of banks allowed. As a general rule, the amount by which the banks are set back, should be such that the slope of the line joining the inner and upper edge of the ultimate berm to the toe of the embankment shall be at least 1 in 6, or better 1 in 7. In special cases where outside borrow-pits are undesirable, the set-back should be such that all the necessary earth for the banks can be obtained from bed borrow-pits not exceeding 4 feet in depth. It will usually not be less than 2·0 feet on each side even in the smallest channels, and may go up to 20 or 30 feet in extreme cases on each side. The amount so set back should be shown on the longitudinal section in the spare column left for this purpose.

8. The height of banks above full supply, even in branches, should not be more than 2·0 feet, after allowing the usual amount for settling of new banks. It is no use piling up height above this; if stronger banks are necessary, put the extra staff into width, not height.

For new distributary banks the height as designed should be 1·50 above full supply. For remodelling old banks 1·0 feet may often suffice, but at the tail 1·5′ should be allowed so as to pass down floods due to sudden closing of outlets.

9. If at all possible, there should be a surface fall of at least one foot from the feeder into the distributary so as to avoid heading up of the former in low supplies. Similarly, there should be a fall into each minor from the distributary of not less than six inches.

10. The surface of flow need not necessarily be parallel to the bed, as the depth may decrease gradually. Changes of section, as regards bed-width, must of course take place at the positions of the " draw-offs," and usually may be by the half foot for small channels and by feet for larger ones. The position and discharge of each " draw-off " must be shown on the longitudinal section in the proper columns, the totals of all the" draw-offs" below any point equalling the calculated discharge there. No allowance need be made for absorption usually, though in special cases it may be necessary.

11. In grading a branch or feeding channel the height of surface flow will be such as to give good command into all the distributaries taking off in that particular reach. It is thus necessary first to fix, approximately, the grading of the distributaries before proceeding to design a branch. Up to now this has seldom been done, even on the newest canals, and the result has been much trouble.

In distributaries it will ordinarily suffice if the water level is kept 1·5 to 2·0 feet above ground-level; there is no advantage in more than this, provided the slope of surface away from the line is fairly favourable.

12. The value of Kutter's N to be used in design should be 0·0225; and side-slopes may be taken as $\frac{1}{2}$ to 1. Longitudinal section scale will usually be two inches to one mile, and vertical scale, $\frac{1}{100}$th.

Sample form of longitudinal section showing data to be given.

	Datum.
	Bed-slope.
	Village.
	Land-width.
	Draw-off.
	F. S. Discharge.
	Velocity.
	Critical velocity ratio.
	Bank-width.
	Ht. of bank above F. S.
	Full-supply depth.
	Bed-width.
	Digging.
	Bed-level.
	N. surface.
	Red: Dist.

PAPER No. 36.

On the remedies being carried out to the Siswan Superpassage Sirhind Canals, to counteract the settlement of the downstream 50 feet.

The Siswan Nulla originally crossed the main line of the Sirhind Canal at reduced distance 7th mile and 2,900 feet. There was no clay in the bed of the Nulla where it crossed the Canal line, so it was determined to move the site for crossing upstream, and the place, 7th mile and 1,500 feet, where the Siswan Superpassage is built was specially selected because it was the only area near the torrent crossing which was covered by a stratum of thick tenacious clay. This clay was 12 feet below canal bed level and 5 to 6 feet deep below the slab of concrete on which it was intended to found the structure. Beneath this clay is fine blue sand of unknown depth, having the characteristics of quicksand, being charged with water under a strong head of pressure, so great, that where the springs burst through the clay they used to rise, when construction of the Superpassage was in progress, to 12 feet above Canal-bed. These springs, wherever they burst, threw up large quantities of fine blue sand, so the most careful precautions had to be taken to prevent the clay being undermined.

It was imperative to preserve the homogeneity of the clay stratum, so well foundations were impossible and the structure was founded on a slab of concrete; the enormous pressure of the piers being distributed evenly over the slab by means of inverts. This slab of concrete was founded all over, 6 feet below Canal bed level. Spring level was kept down by extensive pumping while the concrete was being rammed in position. The Siswan Superpassage was commenced in November 1878 and completed in July 1881.

The Superpassage consists of seven openings of 31¾ feet span at the springing of the arches, and 29¾ feet at the base of the piers. The piers which are 6 feet thick at their upper extremity, and 8 feet at their base, are supported on brick inverts having a versine of 3 feet; these inverts are 2·5 feet brickwork at crown, and thickened up to 3·5 feet under the piers. Directly under the crown of the inverts the foundation slab of concrete is 2·2 feet thick, and is much stronger under the piers. To further insure stability of foundations a curtain of wells was sunk 10 feet below bed, 32½ feet from the up and downstream faces of the structure, and a slab of concrete carried over the intervening space, thus giving a wider spread to the foundations. The reduced level of the torrent bed is 878·0 feet, that of Canal being 854·43 feet, so the torrent bed is 32½ feet above the canal-bed. The width of the torrent-bed between parapets of the Superpassage is 250 feet and the parapets are 16 feet high at upper and 14 feet at the lower end of the work. The Superpassage was originally designed to pass 20,000 cusecs, but since the raising and strengthening of the parapets in 1891 can now discharge 25,000 cusecs. (Plate 52.)

During construction of the Superpassage frequent accidents occurred; these were due to the great head of water that had to be dealt with, and the clay stratum was pierced and damaged in many places. Cracks appeared in the foundations before the work was completed, and numerous others appeared in the superstructure soon after the work was finished. With the exception of a large crack in the left toe path wall which necessitated some reconstruction, none of the cracks in the foundations or superstructure developed up to 1899; they continued with a few exceptions to exist as hair-cracks only.

In 1890 and 1891 it was found necessary to largely increase the waterway of the torrent over the Canal, and the parapet walls were thickened to make them strong enough to be raised and give the needed water-way. The Superpassage-floor was at the same time lowered 1·0 foot by replacing 1¼ feet of pitching by a layer of 3 inches of concrete. The weight of the masonry added to strengthen and raise the parapets was 3¼ tons per foot run of parapet.

In August 1899 Mr. Pargiter, Executive Engineer in charge of the Division, noticed a big transverse crack running right across the structure about 50 feet from the downstream end. He reported that the whole downstream end of the Superpassage had sunk bodily from 4 to 6 inches. (Plate 53.)

In January 1900 when the torrent was in flow the sand on the floor of the Superpassage was washed through this crack which was exposed for the first time from above. In April 1900 Mr. Preston, Chief Engineer, examined the arch cracks during a Canal closure and found that the big transverse crack in the arches followed the joint in two contiguous sections in which the arches were built; this gave the appearance of a very formidable crack. Measurement showed that the downstream face of the Superpassage was slightly out of the vertical. A large number of the stones in the downstream nose of Pier No. 4 had crushed and the out-water was cracked, and showed signs of disintegration due to pressure. The main crack right across the whole work, about 50 feet from the downstream face, was found to extend through the inverts. Mr. Preston, Chief Engineer, recorded that he was unable to say with confidence what had been the cause of the settlement which had occurred. Repair work was confined to prevention of leakage from the torrent-bed into the Canal.

In August 1901 further settlement of the downstream 50 feet was reported and the outward tilt of the downstream parapet had increased, the measurements made showed that the outward tilt of the parapet, which averaged $\frac{6}{32}$nd of an inch from vertical in 1900, averaged $\frac{17}{32}$nd in 1901, or nearly three times as much, so the local officers became greatly concerned about the safety of the work.

Mr. Preston, Chief Engineer, thought that the fears of the local officers were greatly exaggerated, and he declined to have recourse to remedial measures on a large scale until it was known what was the exact cause of the settlement.

It is very curious that although the sinking of the downstream end of the Superpassage was discovered in 1899, yet in the annual closure of the next three years, no careful examination of the floor was ever made, so as to ascertain exactly what might be the cause. Theories were put forward and remedies were suggested in accordance with them. There could be little doubt that the great weight of the masonry parapet must be to a large extent a cause.

The fact that it was only the downstream end that had sunk, and not the upstream end, although both were equally weighted with heavy masonry parapets, suggested, that either scour of the bed downstream must have taken place in the flood seasons, or that there was an underground flow of subsoil-water gradually removing the sand beneath the foundations.

The regular measurements, made every tenth day of silt on the Canal-bed, never showed any scour, but always a foot or more of silt, so no action was necessary as regards preventing scour.

It should be mentioned that after the Canal had been open and running for years, it had acted in the neighbourhood of the Siswan like a huge drain, and reduced the spring level of all the wells in the neighbourhood. When the Canal is open, water rises in the wells to approximately the height of supply in the Canal, and when the Canal is closed spring level all round falls; so that spring level pressure on the inverts when Canal is closed does not now rise to more than 4·0 to 5·0 feet above Canal bed level.

To prevent any underground movement of the sand, if such a thing be possible, a line of sheet piles, tongued into each other, were driven in 1902 right across the Canal-bed, 75 feet from the downstream face of the work.

These piles were driven to 18 feet below Canal-bed level; this left, however an open bed-space of 30 feet between the original downstream bed-pitching and the sheet-piling. At the same time that the sheet-piling was done, designs were prepared for a steel parapet to replace the enormous and heavy masonry parapet on the downstream side of the Superpassage.

The advantages of this are twofold, firstly to remove 6·1 tons per foot run of useless weight, and secondly, to substitute for a rigid and unyielding

material like stone masonry, a parapet of steel which would have a certain amount of flexibility and which would not be seriously impaired even if slight movements in the work should continue.

In the light of knowledge gained about the work up to the end of 1902 the reasons for subsidence then appeared to be—

(*a*) Springs carrying away the subsoil from under the work owing to the pressure of the load of masonry.

(*b*) Diminution of the depth of the impervious layer of clay which opposes the escape of the subsoil-water, by scour of Canal-bed downstream of the work.

(*c*) Actual compression of the layer of clay under the foundations.

At the end of 1902, besides the one line of sheet-piling driven across Canal-bed in that year, the following remedies were proposed:—

(*i*) Removal of the stone masonry parapet and replacing the same by a steel one.

(*ii*) A line of falling shutters to be erected on Canal-bed downstream of the Superpassage to hold up water 5 feet on Canal-bed as a minimum when a Canal closure was on, or when supply was very low.

(*iii*) Stock-ramming the foundations with liquid Portland cement.

(*iv*) Protection of the Canal-bed downstream of the work with a 2-foot layer of concrete to a distance of 75 feet from the work, that is, up to the line of sheet piles.

(*v*) Drainage of the Superpassage at both ends for land springs.

(*vi*) Drainage of torrent-bed on either side over the work.

Early in 1903 the diagonal cracks that had appeared sometime before in the arches of the shore bays, caused so much anxiety that it was decided to support the downstream 50 feet of the arches in these bays: this would prevent the great detached triangle of arch masonry lying between the transverse and diagonal cracks from falling into the Canal and thus wrecking the whole work.

The propping of the downstream 50 feet of the shore bays was arranged to be done with three longitudinal rows of wooden uprights.

In a note, dated 13th March 1903, the Inspector-General of Irrigation (Mr. Sidney Preston) criticised the whole of the remedial measures and approved of *i*, *iii* and *iv*; especially stock-ramming the subsoil material beneath the work with liquid Portland cement so as to fill up all cracks and cavities. He did not anticipate any good from drainage of the work and torrent-bed behind the abutments (*v* and *vi*), as he doubted if the evil was caused by percolation from the torrent flood-waters.

In March 1903 Mr. Benton, Chief Engineer, inspected the Siswan Superpassage and the following works were done during the Canal closure, April and May 1903:—

(*i*) The downstream concrete floor on the Canal-bed was extended up to the line of piles driven in 1902, so that there was a water-tight floor 75 feet below the downstream face of the Superpassage.

(*ii*) All the cracks in the foundations were stock-rammed with liquid Portland cement.

(*iii*) The arches of the side bays were shored up with timber struts for a distance of 50 feet from their downstream ends.

The following is a description of the work done in the closure of 1903. The Sirhind Canal was closed on the 25th April 1903, and as soon after as possible the foundations were unwatered and the floor of the whole work was, for the first time, piece by piece, thoroughly examined, and all cracks located.

During this examination the cause of the subsidence of the downstream 50 feet of the work was for the first time clearly demonstrated by the existence of a very formidable crack in the foundations running right across the structure

approximately along the junction between the brick inverts and the concrete apron downstream. This main crack was nearly under the downstream face of the work. The disintegration of the stone in the downstream pier noses, referred to above, showed that sinking was due to the great weight of the superincumbent masonry, whereas just beyond the downstream face there was no weight, so no cause for subsidence existed.

When this main crack was laid bare nearly all of it was found to be blowing sand silt, shells and even brick ballast in places, so this revealed the cause of the mischief.

It was evident that on every occasion the Canal was closed or a low supply was running and the water pressure over the floor was removed, the pressure from below being unbalanced was blowing up water with sufficient force to carry with it material from under the floor. Evidently this action had been going on for years.

Every time the Canal was closed this blowing out of material must have occurred, causing cavities to be made under the masonry foundations, and, as a consequence, the structure gradually sank to occupy the space from which the sand had escaped.

The tension in the arches, caused by the subsidence of the downstream face, at length caused the last 50 feet portion to break away from the rest, in a long transverse crack, in every bay right across the whole structure; there being at 50 feet from the downstream face a joint of least resistance, which was the construction joint of the arches.

Closing this main crack was started at once. At first it was all covered with fine and coarse ballast mixed, and over the holes where the stock-ramming pipes were to be erected, 3-inch diameter pipes were placed 4 feet high; by this means the top of the crack was temporarily closed, and the escape of sand, etc., was arrested, so there was very little escape of material from under the foundations during Canal closure.

Stock-ramming was a new kind of work, so experience had to be gained as the work proceeded.

Generally the work was done as follows: Bunds were made up and downstream of the cracks that were to be stock-rammed, and before the bunds were closed all the masonry floor was cleared and cleaned completely; the cracks were then covered with fine and coarse ballast.

Holes were jumped on each side of the crack or occasionally in the crack itself; after the holes had been jumped 6 inches deep, 3-inch diameter gas pipes from 3 feet to 4 feet long were inserted into the holes, and tamped outside. These pipes acted as guides, and jumping was continued inside them. When the hole was jumped through to the bottom of the foundations, spring-water burst up and flowed over the top of the guide-pipe, but the hydrostatic pressure was just sufficient to prevent sand, etc., from being blown through.

Two-inch internal diameter gas pipes were then put down reaching to the bottom of the holes, and additional lengths were screwed on to give a head of 20 feet above foundation-level.

The head of pressure was limited to 20 feet of liquid cement and that usually employed was 16 feet. As liquid cement is twice the weight of water, this is equal to a head of 32 feet of water, or 15 lbs. on the square inch. About equal volumes of cement and water were used for the slurry, the actual proportions varied to suit requirements.

The final result of the stock-ramming was to greatly diminish the enormous leakage through the cracks, and in some cases it was stopped entirely.

All the cracks were stockrammed in a similar manner, and the stone-pitching downstream of the concrete floor was covered with 3 inches of cement concrete.

An attempt was made to stock-ram the quick-sand under the layer of clay; it was unsuccessful.

The total quantity of cement used in stock-ramming during the Canal closure, April and May 1903, was: at inner transverse crack 37½ barrels of 4 c.ft. . . . . = 150 c.ft.

at noses of piers or downstream transverse crack 153¾ barrels of 4 c.ft. . . . . = 615 c.ft.

	Total .	765 „

All the holes jumped through the masonry were made with 12 to 14 feet long heavy octagonal steel jumpers.

The second work done in the Canal losure of 1903 was laying a concrete apron 2 feet thick, between where the original downstream masonry floor ended, and the line of sheet piles driven in 1902. This concrete floor consisted of one foot of cement concrete composed of—

	Parts.
Fine ballast	80
Coarse sand	40
Portland cement	20

and on top of the foot of cement concrete one foot of kankar lime concrete composed of:—

	Parts.
Stone ballast 1 to 1½″	110
Kankar lime	18
Surkhi	27

When this concrete floor was being laid, springs in the Canal-bed were found very troublesome. Also two iron drainage pipes were discovered 2 feet below Canal-bed; both these burst open, throwing very large quantities of water in which was some sand.

Tubes were fixed into the ends of these, and the water was allowed to flow away over the completed concrete till the latter had set hard.

One of these drainage pipe springs was closed with liquid cement, but the largest one opposite the 4th Bay had to be left flowing, as the concrete floor had not set firm enough round it to admit of the pipe being closed by the time the Canal was re-opened. The springs in the Canal-bed were allowed to run while the concrete floor was made all round them.

After it had set, the springs were surrounded by brickwork wells, and allowed to rise to their full height, and then plugged with a foot of cement concrete.

The third work done in the Canal closure of 1903 was the shoring up, with deodar timber, the downstream 50 feet of the arches of Left and Right Bays.

This work was done in order to support the triangular cracked portions of those two arches which threatened to drop down into the Canal.

The shoring consisted of three longitudinal rows of timber struts 12 inches by 6 inches section, all properly strutted to prevent horizontal movement.

These were placed 18 inches apart centre to centre under that portion of the arch carrying the parapet, and 3 feet apart for the remainder of the length.

The erection of these frames was begun as soon as the tamping of the invert cracks made it possible to unwater the shore bays. Finally the diagonal arch cracks were carefully grouted with cement to restore contact, and the horizontal cracks in the downstream parapet wall were also filled with cement.

That finished the work done on the Siswan Superpassage in May 1903.

The steel parapet to replace the downstream masonry one was then being constructed at the Amritsar Workshop, and it was intended to erect it immediately after the rains; that would relieve the foundations of an enormous load. (Plate 54.)

Also in order to strengthen and prevent the floor shearing again along the downstream face cracks, designs were being worked out to extend the piers and inverts downstream so as to make the pressure diminish gradually, instead of abruptly as is now the case, as the inverts end just beyond the downstream face of the Superpassage. Early in September 1903 constructing the steel parapet was completed and it was temporarily erected in the Amritsar Canal Workshop-yard, it was then taken to pieces, and despatched to Doraha Station, North-Western railway, whence it was taken in boats up the main line of the Sirhind Canal to the Siswan Superpassage.

A temporary bund was made in the torrent-bed parallel to the downstream parapet wall. This bund terminated up and downstream against the torrent wings of the Superpassage. By means of this bund the downstream parapet was isolated, so the demolition of the masonry parapet and erection of the steel one in its place was possible; there was still 200 feet torrent-bed width left, and this allowed ample water-way for any cold weather flood-water to still pass over the work, and find its way into the Sutlej river.

Dismantling the downstream parapet was done evenly over the whole length of the wall, each layer of masonry being taken off one after another. A perfectly level layer of ashlar masonry was constructed, on which the positions of the anchor bolts of the steel parapet were marked and the holes jumped to take the anchor bolts. Before the anchor bolts were cemented in position the steel frames to which the parapet sheeting was to be rivetted were erected, and fastened together temporarily by their angle irons and bolts. Portland cement was then put in the anchor bolt holes and they were allowed to set; proceeding in this way, there was no chance of mistakes being made in the position of the anchor bolts.

The steel frames were then removed, and the anchor bolts tested up to the full limit of the greatest stress they might have to bear.

As the steel parapet was 268 feet long, expansion and contraction had to be arranged for. So the steel frames, which are three feet apart centre to centre, and which carry the iron sheeting of the parapet, were erected on sill beams. There are twin channel iron sill beams just below the inner torrent edge of the steel parapet, and one sill beam under the downstream ends of the parapet.

These sill beams were anchored securely to the masonry parapet, and the steel frames were bolted to the sill beams.

The bolts fastening the steel frames to the sill beams are not screwed up too tight, except the bolts of the central frames; these were secured as tight as possible, the bolts of the frames on either side were left slightly loose, thus expansion and contraction of the whole structure could, and does, take place from the centre to the sides.

The erection of the steel parapet replacing the masonry one was completed in March 1904.

Early in the cold weather of 1903-1904 specimens of stone from different railway and canal quarries in the Punjab were sent to the Mechanical Laboratory, Sibpore Engineering College, where the crushing load for each class of stone in tons per square inch was found by experiment.

Taraki quarry stone, which gave an average ultimate crushing load of 3·70 tons per square inch, was selected.

In March plans were sanctioned and arrangements made with contractors to commence quarrying and dressing ashlar stone for the superinverts and buttresses to be erected partly under, and partly downstream of the Superpassage arches, and on the old inverts. These stone superinverts in continuation of the present ones will distribute the pressure of the superincumbent load of masonry, and by means of buttresses they will be made to bear part of the weight. Also the new inverts will cover the main downstream cracks, and so close permanently the leaks and springs. The only difficulty is to get the buttresses to act as continuations of the piers. Mr. Kennedy, Chief Engineer, has arranged for this by means of four rolled beams acting as struts, in the centre of each buttress. These will be placed under the arch springing at

each pier, and will be held back at their lower extremities, by a tie on each face of the pier.

These superinverts and buttresses will consist of the best stone masonry in Portland cement mortar, and the buttresses will be bonded into the downstream face of the Superpassage.

The work that was done to the Superpassage in the annual Canal closure of May 1904 was as follows:—

(*a*) All cracks were again stock-rammed, especially those that were found open and leaking.

(*b*) Three-inch diameter stand-pipes were erected at the downstream ends of abutments and at noses of piers, to gauge what the real head of spring pressure is below the foundations.

(*c*) Five out of a total seven superinverts were laid to haunch level in 2nd, 3rd, 4th, 5th, and 6th Bays; the stone for the remaining two Bays was not ready on the 29th April 1904 when the Canal closure commenced.

Previous to the Canal closure some experiments were made to discover the effect of stock-ramming, and the time taken to set. It was found impossible to make liquid cement penetrate wet sand.

The slurry usually set in from 24 to 48 hours. A mixture of about 20 per cent. by volume of carbonate of soda to 80 Portland cement was used for tamping round the guide-pipes and between the 3 inches guide and 2½ inches pipes put inside them to take the cement-slurry; this mixture set almost at once and was suggested by Mr. Kennedy, Chief Engineer, and was of great assistance in making good joints between the floor, and the pipes down which the cement slurry was poured into the foundations.

After the Canal was closed, early in May 1904, the Bays were unwatered, one at a time, and examined, the condition of the invert and floor cracks was found to have improved.

The 5th, 6th, and 7th Bays were practically watertight, but the iron tube spring in the concrete floor downstream of the 4th Bay was found flowing vigorously and blowing sand. The downstream face main crack was leaking badly, and blowing sand in the 2nd, 3rd, and 4th Bays, and slightly in the 1st Bay. There was no leakage from the invert 50-foot crack, except slightly in 1st and 3rd Bays.

The water was immediately headed up by means of bunds, to prevent the escape of sand from the foundations in those Bays where the cracks were leaking.

It rose highest in the 4th Bay to R. L. 859·29, that is, 4·8 above Canal-bed level.

At the same time guide-pipes were erected on either side of the main crack and holes bored down to foundation-level in which 2½-inch pipes were erected 18 to 20 feet above Canal-bed for stock-ramming. A steady continuous flow into each pipe of cement slurry was arranged for from a double compartment tank placed over each pipe at the required height above the Canal-bed. These tanks were supported on a scaffolding of bamboos and planks.

No stock-ramming was started in a Bay until the spring water had risen and remained standing at its highest level for from one to two days.

Also all cracks were closed on the surface at invert level with a mixture of carbonate of soda and cement, which set at once, so there was no chance of any of the slurry escaping after stock-ramming commenced. In fact, every precaution was taken to make the stock-ramming successful, and not lose cement by the slurry escaping in the running water of leaks and springs.

Stock-ramming commenced in the 6th Bay. As the main crack under the downstream face in that Bay was practically not leaking, only one hole was sunk at first and one pipe for stock-ramming was erected. This pipe took 16

barrels of cement, and after some time the slurry began to appear at the tube spring downstream of the 4th Bay; the flow of the spring gradually decreased, as stock-ramming continued, and at last ceased entirely.

To prevent this pipe spring bursting out again, a ring bund of earth was made round it and water was headed up over it. The slurry from No. 6 Bay also appeared in the invert cracks at No. 4, and slurry from 2nd Bay appeared in the pipes of the 3rd and 4th Bays, showing there is a passage or communication under the foundations between one Bay and another.

When all the arrangements were complete for stock-ramming in the 2nd and 4th Bays, and the water was headed up fully over their main cracks, also over where the spring pipe had ceased to flow in the concrete floor, suddenly an enormous spring burst out in the bed of the Canal 47 feet downstream of the line of sheet-piling.

The crater of this spring was at once treated with fine pea ballast covered with coarser ballast. An area of 10 feet by 10 feet was covered, and weighted down with rubble stone; however, the spring did not stop, and remained running when the Canal re-opened on 25th May 1904.

This shows that as the floor and inverts were made watertight, the spring-level pressure on the foundations forced a passage under the 75-foot length of concrete floor and then appeared in these springs further down in the Canal-bed. Two more small ones afterwards developed in the Canal-bed downstream of Bays 1 and 5.

By the time stock-ramming was finished and the five superinverts were laid, there were no leaks in any but the 1st Bay, where a few small ones were still visible; these will all be finally closed when the downstream end of the shoring timbers are removed, and the new superinvert laid.

One hundred and four barrels of Portland cement of 4 cubic feet each, 416 cubic feet in all, were used in stock-ramming the cracks and leaks in the inverts and floor downstream during the closure of 1904.

The stand-pipes were erected at the ends of the downstream piers and before and since the re-opening of the Canal in May 1904, the spring level pressure on the foundations has been observed.

Of the remedial measures that have been sanctioned, but not yet carried out, there remain the laying of the superinverts in the 1st and 7th Bays; building the invert abutments on either side of the Canal, and the buttresses at ends of each downstream pier. Probably also the length of the watertight floor downstream of the Superpassage on the Canal-bed will be increased considerably.

Plate 55 shows the height of water in the stand-pipes observed on 25th May 1904 after the Canal had been closed for 25 days. Even after that long closure there was an average pressure of :—

Average level of water in stand-pipes.	Level of water on Canal bed.	Head of water-pressure.
859·95	855·43	4·52 feet.

Floods only last a few hours; immediately after a flood has passed the bed dries up and spring level falls to 4 or 5 feet or more below the bed of the torrent.

The Canal was closed on the 5th July and kept closed till the evening of the 6th. Spring level was observed in the stand-pipes on the 6th July while the Canal was closed and the average level is nearly the same as that of the 25th May.

The diagram shows clearly that the most dangerous time is when the Canal is closed; the water-pressure on the foundations is such that if it finds an exit, it must carry with it the material from underneath the floor.

The level of the water in the stand-pipes is always slightly above the level of supply in the Canal; however, it is doubtful if the small difference in level would be sufficient to cause the springs to work when the Canal was running.

When a flood is passing over the Superpassage no rise of level has been noticed in the stand-pipes in the Canal, so there does not appear to be any direct communication or percolation between the flood-water of the torrent and spring-level of the Canal-bed.

As to the exact state of the downstream 50 feet of the Siswan Superpassage at present, I think conclusions can best be drawn from the measurement of the cracks. These have been observed very accurately every month since early in 1900.

It was quite evident when Mr. Field, Chief Engineer, wrote his inspection note, dated 22nd April 1902, that the cracks had increased from $\frac{1}{2}$ to $\frac{3}{4}$ inch between February 1901 and January 1902.

Also from the attached statement the cracks were still increasing till after the Canal closure of 1903, during which the first real attempt was made by stock-ramming, etc., to close up the leaks and springs in the floor of the inverts and in the bed of the Canal downstream of the Superpassage.

Since the Canal closure of 1903 up to date the measurements of the cracks have varied, sometimes showing they have slightly increased, sometimes slightly decreased. However, from having actually personally made the measurements I know it is impossible to measure more accurately than to the $\frac{1}{32}$nd of an inch. Even that accuracy is not attainable by the same measurer two days running; so a difference of 0·03 inch does not indicate that the crack has increased or decreased that much, but simply that either the same measurer, or another, did the work on two different dates.

If what I state above is correct, and I believe it is, then the measurement of the cracks from June 1903 to June 1904 show that the last 50 feet of the Superpassage as a whole has ceased to subside.

I do not say that all movement has ceased, as new hair-cracks have appeared in two or three new places, but that the annual rate of subsidence has diminished to so little, that it can no longer be detected by measuring the width of the cracks.

If this improved state of things continues for another year, and all the remedies that have been sanctioned and are being carried out, have been completed, then I think a reliable opinion can be formed as to whether the subsidence of the downstream 50 feet of the Superpassage has ceased or not.

If subsidence still continues, then, as Mr. Kennedy, Chief Engineer, suggested, the steel parapet will have to be set back, say, 60 feet upstream of the present parapet, and connected to the shore wings on the left side of the Superpassage at each end.

When the continued subsidence makes the use of the downstream 50 feet no longer possible, then the arches would be removed and probably some of the piers, and the whole downstream 50 feet would be replaced by an iron-work structure of light steel girders and trough flooring. The present steel parapet would, of course, be used, so the money spent on it would not be wasted.

Whether this alternative will have to be undertaken or not, is at present doubtful. However, the cracked arch triangular portions of Bays 1 and 7 must be rebuilt soon; probably next year they will be renewed.

Comparative statement showing measurement of cracks in Siswan Superpassage (Difference) for 1901-1904.

Number of cracks.	Particulars of measurements, marks regarding arches counting from Left Bank.	Measurement of cracks on		Difference in decimals of an inch.	Measurement of cracks on		Difference in decimals of an inch.	Measurement of cracks on		Difference in decimals of an inch.	Remarks.
		16th February 1901.	28th January 1902.		28th January 1902.	28th June 1903.		28th June 1903.	6th July 1904.		
1	2	3	4	5	6	7	8	9	10	11	12
		Inches.	Inches.		Inches.	Inches.		Inches.	Inches.		
1	1st Arch	10·83	10·92	*0·09*	10·92	11·01	*0·09*	11·01	11·00	0·01	
2	Do. (Centre)	18·27	18·36	*0 09*	13·36	18·53	*0·17*	18·53	18·54	*0 01*	
3	Do.	18·02	18·05	*0·03*	18·05	18·21	*0·16*	18·21	18·25	*0·04*	
4	2nd Arch	17·70	18·20	*0·50*	18·20	18·28	*0·08*	18·28	18·25	0·03	
5	Do. (Centre)	19·27	19·37	*0·10*	19·37	19·58	*0 21*	19·58	19·60	*0·02*	
6	Do.	13·64	13·65	*0·01*	13·65	13 68	*0·03*	13·68	13·63	0·05	
7	Do.	13·15	13·17	*0·02*	13·17	13·18	*0·01*	13·18	13·14	0·04	
8	3rd Arch	23·32	24·07	*0·75*	24·07	24·25	*0 18*	24·25	24·15	0·10	
8 A	Do. (Centre)	19·50	19·50	...	19·50	19·73	*0 23*	19·73	19·72	0 01	
9	4th Arch	13·36	14·06	*0·70*	14·06	14·28	*0·22*	14·28	14·29	*0·01*	
10	Do. (Centre)	20·07	20·90	*0 83*	20·90	21·27	*0·37*	21·27	21·27	...	
11	Do.	20·28	20·98	*0·70*	20·98	21·30	*0·32*	21·30	21·34	*0·04*	
12	5th Arch	20·35	21·10	*0·75*	21·10	21·56	*0·46*	21·56	21·58	*0·02*	
12 A	Do.	12·70	13·41	*0·71*	13·41	13·73	*0·32*	13·73	13·79	*0·06*	
12 B	Do.	14·38	15·02	*0·64*	15·02	15·38	*0·36*	15·38	15·41	*0·03*	
13	6th Arch (Centre)	18·89	19·50	*0·61*	19·50	20·00	*0·50*	20·00	20 10	*0·10*	
14	7th Arch (Centre)	17·58	17·57	0·01	17·57	...	...	...	...	...	
15	On right side of Arch.	20·86	20·92	*0·06*	20·92	21·25	*0·33*	21·25	21·47	*0·22**	* This measurement is of the triangular detached piece of No. 7 Bay Arch; along one side crack is decreasing: and on the other increasing, only shows its weight is resting more on the shoring up timbers.
16	7th Arch (Centre)	19·51	19 51	...	19·52	...	Under shoring timbers				
17	6th Arch	14·98	15·10	*0·12*	15·10	15·17	*0·07*	15·17	15·05	0·12	
18	7th Arch	12·62	12·62	...	12·62	12·78	*0 16*	12·78	...	...	
19	Do.	20·22	20·22	...	20·22	20·26	*0·04*	20·26	20·28	*0 02*	
20	Do.	9·95	9·95	...	9·95	...	...	...	...	...	
21	1st Arch	17·44	17·47	*0·03*	14·47	...	Under shoring timbers				
22	Do.	18·01	18·10	*0·09*	18·10	...	...	...	...	...	
23	D. S. F. L. A.	8·50	8·49	0·01	8 49	8 50	*0·01*	8·50	8 50	...	
24	1st Arch	13·58	13·70	*0·12*	13·70	13·69	0·09	13·69	13·61	0·08	
25	Do.	*Note* Three	16·12	...	16·12	16·13	*0·01*	16·13	16·10	0·03	
26	Do. (Centre)	cracks were	14 05	...	14·05	...	...	...	...	...	
27	Do.	found on 30th	16·72	...	16·72	16·75	*0·03*	16·75	16·68	0·07	
28	7th Arch	June 1901	10·70	...	10·70	10·78	*0·08*	10·78	10·75	0·03	
29	Do. (Centre)	and were *Nil*	13·75	...	13·75	...	...	...	...	...	
30	Do.	measured on 30th July 1901.	13·05	...	13·05	13·50	*0·45*	13·50	13·57	*0·07*	

N.B.—Figures in Italics show increase.

Statement showing measurement of the cracks in the Siswan Superpassage.

Number of cracks.	Particulars of measurement marks regarding arches counting from Left Bank.	1903.					1904.					Remarks.
		June.	August.	September.	October.	December.	January.	March.	April.	May.	July.	
		28th.	19th.	3rd.	26th.	2nd.	2nd.	1st.	2nd.	2nd.	6th.	
		Inches.	Inches	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	
1	1st Arch . .	11·01	11·04	11·00	11·01	11·00	11·00	11·00	11·00	11·00	11·00	
2	Do. (Centre) .	18·53	...	18·53	18·55	18·55	...	...	...	18·50	18·54	
3	Do. .	18·21	...	18·24	18 25	18·25	...	...	...	18·28	18 25	
4	2nd Arch . .	18·28	18·24	18·25	18·22	18·20	18·19	18·18	18·18	18·26	18·25	
5	Do. (Centre) .	19·58	19·57	19·61	19·61	19·60	19·62	19·60	19·60	19·58	19·60	
6	Do. . .	13·68	13·63	13·63	13·64	13·63	13·62	13·63	13 60	13·63	13·63	
7	Do. .	13·18	13·13	13·14	13·13	13·13	13·13	13·12	13·15	13·21	13·14	
8	3rd Arch . .	24·25	24 24	24·22	19·77	24·19	24·19	21·15	24·12	24·12	24·15	
8A	Do. (Centre) .	19·73	19·72	19·73	24·15	19 75	19·76	19·78	19 75	19·70	19·72	
9	4th Arch . .	14·28	14·28	14·29	14·28	14·28	14·30	14·30	14·29	14·26	14·29	
10	Do. (Centre) .	21·27	21·32	21·35	21·33	21·34	21·35	21·35	21·30	21·25	21·27	
11	Do. .	21·30	21·37	21·34	21·34	21·37	21·37	21·38	21·35	21·31	21·34	
12	5th Arch (Centre) .	21·56	21·62	21·65	21·63	21·63	21·63	21·63	21·57	21·58	21·58	
12A	Do. .	13·73	13·69	13·78	13·78	13·80	13·80	13·81	13·77	13·78	13·79	
12B	Do. .	15·38	15·42	15·44	15·45	15 46	15·46	15·49	15·45	15·46	15·41	
13	6th Arch (Centre) .	20·00	20·10	20·08	20 12	20 16	20·18	20·20	20·10	20·10	20·10	
14	7th Arch (Centre) .	...	...	...	...	...	...	Under shoring timber.				
15	Do. .	21·25										
	Side of arch triangular piece.		21·40	21·41	21·46	21·53	21·50	21·50	21·51	21·52	21·47*	* This is at end of triangular detached Arch piece.
16	7th Arch (Centre) .	...	...	...	...	...	...	Under shoring timber.				
17	6th Arch . .	15·17	15·10	15·10	15 08	15·09	15·08	15·10	15·07	15·06	15·05	
18	7th Arch . .	12·78	12·71	...	12·71	12·75	12 74	12·74	12·73	12 71	12·73	
19	Do. . .	20·26	...	20 20	20·20	20 26	20·23	20·25	20·22	20 24	20 28	
20	Do. . .	...	...	...	9·97	10·00	10·00	9 99	9·94	9·96	9·96	
21	1st Arch . .	...	...	...	17·50	17·52	17·52	17·55	17·50	17·50	17·48	
22	Do. . .	...	...	...	...	18·55	18·58	18 60	18·56	18·51	15·50	
23	D. S. F. L. A. .	8 50	8·49	8·47	8·50	8·50	8·51	8·50	8·50	8·50	8·50	
24	1st Arch . .	13·69	13·68	13·67	13·63	13·65	13·61	13·64	13·61	13·61	13 61	
25	Do. . .	16·13	16·13	16·10	16·09	16·10	16·09	16·13	16·09	16·10	16·10	
26	Do. (Centre) .	...	...	...	...	...	...	Under shoring timber.				
27	Do. .	16·75	16·70	16·72	16·70	16·69	16·70	16 69	16·67	16·69	16·68	
28	7th Arch . .	10·78	10·73	10·73	10·77	10·74	10·75	10·70	10·71	10·71	10·75	
29	Do. (Centre) .	...	...	...	...	...	...	Under shoring timber.				
30	Do. .	13·50	13·55	13·57	13·57	13 65	13·65	13·67	13·61	13·58	13·57	

PAPER No. 37.

On Lessons to be learnt from Sirhind Canal silt trouble.

NOTES ON—

RESULT OF SILT EXPERIMENTS AND SILT CLASSIFICATION.
USELESSNESS OF HIGH SILLS AS SILTING PREVENTORS.
BEST METHOD OF WORKING HEADWORKS.

In a separate note, following this, Mr. Gillmon has given a description and history of the Rupar Headworks. About 1893 the Sirhind Canal threatened to silt up at the head altogether: numerous remedies were suggested: some were carried out, and also many experiments on this silt question. Hitherto the results of these measures have not been analysed, and though Punjab *Irrigation Paper No. 9 gives a great mass of figures, yet nothing definite is arrived at, and every reader is left to form his own conclusions. This memorandum aims at supplying this need—partially at any rate.

Silt experiments and general results of any value.

The silt experiments, extending from 1893 to 1897, consisted in pumping up from different depths, at several points in the Canal, a certain quantity of water with its contained silt, then evaporating the water and weighing and measuring the sediment. It was not then recognised, however, that our chief trouble is with sand or bed-silt, some of which partly rolls on the bed or, at any rate, never rises high above the bed, and consequently would and did escape notice in these experiments, which are therefore to a certain extent vitiated. True a certain amount of the coarser sand was found in the higher layers of water, but here, again, no correction was made for this sand coming up in the pipe at a less rate than the containing water. This purely rolling sand is not included in the figures given below, and we have no *data* to show how much it might be. The only way to find this would be to trap it as it rolls in a pit or orifice of some sort: it should be quite possible, but has never been done.

Up to 1897 the experiments included *all* sediment in the water, and the proportion of this to the water by volume was found to vary immensely, *viz.*, from zero up to somewhat about a maximum of $\frac{1}{300}$th at the Canal-head. Most of this latter was fine clay or ooze, which would take some time to settle and would give no trouble in the Canal; part of it was sand of various grades—how much was not observed, probably not more than one-third. We cannot make much of the *data* thus collected; no relation between depth and silt ratio could be arrived at.

In 1898 a sort of sand separator was devised which though rather crude undoubtedly separated out the greater part of the sand down to the finest grades for measurement and carried off most of the fine mud which was not measured at all. However, the results were a great advance on the previous experiments. Analysing these I find that in the flood season the proportion of *sandy* sediment entering the Canal-head was seldom over $\frac{1}{1400}$th, and only once went up to $\frac{1}{1000}$th; the average for river-floods may be taken as $\frac{1}{1700}$th. At the same time the proportion carried off by the Canal-water 26 miles down from the head varied from about $\frac{1}{4000}$th to $\frac{1}{2800}$th, and the average was roughly $\frac{1}{3300}$th. The difference, therefore, between $\frac{1}{1700}$th and $\frac{1}{3300}$th, or $\frac{1}{3300}$th the volume of the Canal-supply, or half the whole sandy silt carried, was left on the Canal-bed during the times of high river-flood. This figure has some weight, and means that as long as the water entering is fully charged with sandy sediment from finest sand to the coarsest, then if the Canal

* Not reproduced.

has been designed to have what is now called the "critical velocity" (*i.e.*, just keep its silted bed clear all the year round) it will only be able to carry off $\frac{1}{3300}$th of its volume of supply in *suspension* and any excess of sandy silt will be left on Canal-bed. This, it must be remembered, leaves out of consideration all fine mud and rolling sand on the bed. What proportion it could carry off including these is doubtful; it will certainly be very much more: canal silting reach experiments usually show that about $\frac{1}{2000}$th of the supply is deposited, but this includes both flood and comparatively clear water, and often the whole sediment is not dropped.

Up to early September the river-supply is usually fully silt-charged with both clay and sand, and deposits of the latter continue to accumulate on the head reaches of the Canal. After that the water clears, and is able to pick up and carry forward the bed-silt, the finer sand going on faster than the coarse; so that the longer clear water has been running the coarser is the sand on the bed near Canal-head. Bye and bye there is thus an insufficient supply of the finer sand to fully charge the Canal-supply farther down, so that instead of carrying $\frac{1}{8300}$th of its volume in *suspension* as in August, it only carries about $\frac{1}{9000}$th in November and December. That this sand is really of a coarser grade than before is fully shown by analysis and we are quite justified in concluding that a canal in permanent *régime* (with critical velocity V_o) will carry sand off in *suspension* in proportions varying from $\frac{1}{8300}$th to $\frac{1}{9000}$th according as its supply is fully charged with all grades of sand or only with the coarser grades. It should be added that this proportion does not seem to vary much, if at all, with the supply in the Canal.

Sand silt classification.

It will be now quite plain that until we can classify and separate all silt we are working in the dark. This is quite possible, and I would refer to a classification proposed by me, based simply on the rate of falling in water. Thus sand that fell at the rate of 0·10 foot per second would be of grade $\frac{}{0\cdot10}$; all sand included between grades $\frac{1}{0\cdot10}$ and $\frac{1}{0\cdot20}$ would be called—class $\frac{0\cdot10}{0\cdot20}$. A diagram showing the results of experiments in this directon is given in Plate 57 and shows by curves how greatly sand silt from various places differs. Thus the steeper the curve the coarser the sand, and the remarks given above as to the bed-sand being coarser nearer the head of Canal than farther down will be seen to be fully justified. The same action is seen in the two curves of the River Sutlej bed at Rupar, and at Ganda Singhwalla (about 150 miles apart). The Chenab River sand comes out coarser than the Sutlej. In general it is seen at a glance that only grades coarser than grade $\frac{1}{0\cdot10}$ stay on the Canal-bed; so that in any future experiments we may eliminate all below this grade.

Sand separator proposed.

To do this the separator used in the Sirhind Canal experiments of 1898 will not answer; Plate 58 shows a design proposed by me in 1898. It merely ensures that the water passing away shall have a vertical velocity just equal to the rate-of-fall of the finest grade of sand which is to be measured, all finer grades passing off in the exit-pipe, which acting as a syphon can have the velocity of flow varied to any desired figure by altering the length of the out-flow pipe. The coarser sands fall into a removeable glass cylinder at the bottom.

Returning now again to Appendix A of Paper No. 9 and eliminating what are obvious errors, we may arrive at some figures as to how much a canal in *régime* (at Garhi in this case) can carry off of this dangerous sand above grade $\frac{}{0\cdot10}$. The last monsoon flood in the river was on September 9th, and up to then we may consider the supply as distinctly muddy. Now up to then I find that the proportion of this coarser sand carried at Garhi was roughly on the average about $\frac{1}{35000}$th of the supply.

From 9th September to 19th or 20th the river went down rapidly, and from then onwards very gradually with fairly clear water. Omitting this interval, as being transitional, I find that—

From 22nd September to 7th October the proportion carried was $\frac{1}{15000}$th.

From 8th October to 23rd October ditto ditto $\frac{1}{32000}$th.

From 24th October to 8th November ditto ditto $\frac{1}{41000}$th.

From 9th November to 24th November ditto ditto $\frac{1}{85000}$th.

These figures, as far as they go, show that much less coarse sand is carried in very muddy water, probably because of the increased viscidity and energy required to carry the mud. This being equally applicable to the river would seem to show that the period *before* the rains should be the most harmful, and I see in the printed reports of 1893 that it was in fact contended that almost the whole silting of bed took place in the snow-water period, though this reason was not then recognised.

The mere appearance of the water therefore shows nothing.

From October onwards the proportion carried again steadily decreased from its maximum of $\frac{1}{15000}$th simply because there was less of the lower class $\frac{0.10}{0.20}$ left on the bed to be picked up.

Remedies adopted in 1893 and their results.

The final outcome of this scare in 1893 was that four remedies were carried out in the winter of 1893-94 as follows:—

(I) Increasing escape power at the 12th mile.

(II) Closing off during heavy floods.

(III) Constructing a divide wall A B parallel to regulator face so as to form a pond or silt trap which should be scoured out occasionally through the under-sluices—see Plate 59.

(IV) Raising a sill 7 feet high at Canal regulator and a movable sill capable of rising still higher, with the idea that only top water, comparatively clear, would be taken in.

Remedy (I) did a certain amount of good no doubt, but as there was seldom any spare clear water it was limited.

Remedy (II) also did good, but it is very likely that the Canal was often closed unnecessarily, simply because the water looked muddy.

Remedy (III) was undoubtedly on the right lines, and the intention evidently was to keep the under-sluices generally closed, so that only the supply really used enters at B. Any regulation would therefore have to be done by the shutters, preferably at the other end C. Unfortunately, however, this was never clearly laid down, and the result was that up to June 1900 the under-sluices were generally more or less open, and it is noted that the leakage through these sluices was often as much or more than the Canal-supply. This of course to a very great extent, or wholly, vitiated the good effects of the "pond," as the water passing through the latter would have a very high velocity, carrying a large proportion of sand, and might in fact sometimes be more silt-laden than the river itself in its normal channel.

It seems rather extraordinary that this was not perceived, as almost self-evident, but facility of regulation by the under-sluices rather than by the shutters may have had something to do with it. Be this as it may, only from June 1st, 1900, was regulation done by the shutters at C and under-sluices kept closed. During that summer, however (as I learn from the Superintending Engineer), it is very doubtful if this was strictly enforced; there being a difficulty in closing all the under-sluice gates for one season. From 1901 inclusive there is no doubt that this manner of regulation was enforced, and it only needs a

glance at the following statement of silt on Canal-bed to show a most marked effect :—

Statement showing Sand Silt deposited on Sirhind Canal-bed Main Linen the first 12 *miles, in lakhs of c.ft.*

	1892.	1893.	1894.	1895.	1896.	1897.	1898.	1899.	1900.	1901.	1902.	1903.	1904.
Maximum (about August).	284	202	187	218	211	249	192	262	208	82	56	67	...
Minimum (about spring).	180	107	100	68	79	73	67	66	70	24	24	16	14
Difference deposited each year.	104	95	87	150	132	176	125	196	138	58	32	51	...

In 1893 the Canal was closed part of the summer, and 1894 was a year of very slight demand, so that frequent and long closures occurred. From these figures we are forced to the conclusion that all the four remedies of 1893 did no good at all as far as the quantities deposited each year are concerned—rather the reverse; but that the scouring was more efficient after 1894, probably because the winter demand increased as cultivation extended. The mere change in the system of regulation has decreased the yearly deposits from about 150 lakhs to 50 lakhs.

Remedy (IV).—That raising the sill did no good is shown without doubt just above, and that this might have been expected is, I think, also clear, when we consider what happens when flowing water strikes against a wall or weir.

All the layers are of course flowing forward, not merely the top, where the weir overflow is; the upper films are deflected upwards and over the crest; the lower films are deflected downwards, strike the bed there, are deflected backwards, and then upwards on meeting the advancing current, thus again mingling with the general stream. That this is so can be seen by looking at the hole scoured out in front of any vertical weir wall, with a silted bed upstream. The velocities of these deflections will not be very much less than the onward flow, which is seldom under 2 or 3 feet per second, and as we have seen that there is hardly any sand coarser than grade $\frac{1}{0.10}$, it is very evident that all sand will be carried over the weir, being lifted up by the upward final deflection. In the case of boulders it would be different as a very strong upward current would be needed to lift them, but in the case of sand no good whatever can result from a raised sill, unless the approach velocity can be reduced to a generally impossible value.

Lessons to be learnt.

The only way to keep silt out of a canal is, therefore, to take out from comparatively still water, depositing all excess sand in the river-bed by ponding up the river. For this purpose the most effective method is to head up the whole river, and do any regulation required at the far side of the weir, trusting to occasional floods to scour out this pond before it has silted up to the safe limit. The defect in this lies in the possibility that such floods may not be sufficiently frequent or high enough to scour out more than a narrow channel, and that not necessarily in front of the head regulator, where we wish specially to scour. In such cases the Divide wall would come in useful, as all the river-supply could be concentrated through it and the under-sluices, ensuring the maximum rate of bed erosion in the limited time often available. Such a divide should not be too short—1,000 feet at the least.

To facilitate regulation it would be well to have some under-sluices at the far side of the river also—at C in plan (Plate 59); this would not be all extra expense, as the flood waterway being increased the weir could be of less length.

It is very important to know when the safe limit of deposits on the river-bed has been reached, so as to arrange for a timely canal closure and river scour. This could, no doubt be shown by a sand-separator, once we can fix

the safe proportion of coarse sand the Canal can carry off. So far as the figures available go, this would, as shown above, seem to be under the most favourable circumstances, for a canal with critical velocity, about $\frac{1}{15000}$ th of sand coarser than grade $\frac{1}{0{\cdot}10}$ but at the best this would be a troublesome process and liable to errors.

A simpler plan is available—for we know that if we consider any number of channels fully silt-charged on silted beds, then so long as the mean velocity of each is a certain function of the depth (*viz.*, V_o-$0{\cdot}84d^{0{\cdot}64}$), the silt proportion carried will be about the same. As soon therefore as the velocity in the river's approaching current exceeds for its depth this value of mean velocity (Vo), it shows that more sand is entering the canal than it can carry off. Thus if the depth in the divide or river in front of the regulator was 8 feet, its mean velocity should not *exceed* 3·18 feet per second. Of course often the depth will vary at different points, and there the observations must be made where soundings show a fairly uniform section, and at several points; a velocity meter would be the simplest means, lifting it uniformly from bed to surface, thus reading off the average velocity direct. The following table of these critical velocities for different depths is given for ready reference, as found true for Punjab canals, and presumably also true for Punjab rivers, were it possible to establish on them a permanent *régime* :—

Depths .	1	2	3	4	5	6	7	8	9	10	15	20	30	40
Critical velocities V_o.	0·84	1·3	1·70	2·01	2·35	2·64	2·92	3·18	3·43	3·67	4·75	5·70	7·40	8·90

Note on arrangement of gates at Rupar Head Regulator, with large openings and the lower gate rising and falling behind a raised sill.

A brief description of the head-works of the Sirhind Canal will aid in following the causes and reasons which led to the adoption of the subject of this paper. The Canal head-works are situated on the River Sutlej at Rupar, in the Ambala District, Punjab, where the Sutlej has a bed-gradient of about two feet per mile, and where its bed is composed of boulders and shingle on which is super-imposed sand. The minimum discharge of this river is some 3,000 cubic feet per second, while the maximum flood-discharge has been computed to be about 135,000 cubic feet per second. The melting snows cause the river to rise early in May, the heaviest floods occur in July and August, and by October the flood season is over.

The under-sluices are at right angles to the regulator and also to the river, but the weir, which is 2,399 feet long, inclines from the under-sluices at an angle of 15 degrees upstream. The right flank of the weir is a revetment wall set against the high cliffs, which at that point form the bank of the river. The level of crest of the present weir is R. L. 866·30 and that of top of shutters is R. L.872·30. The general arrangements and relative positions of the head-works at Rupar are shown on Plate No. 60, as they existed in 1903-1904. The under-sluices consist of twelve openings each of 20 feet span, with three iron draw-gates in each vent, an upper, middle and bottom gate, each working in separate contiguous grooves. These gates are lifted and lowered by a traveller running on rails laid along the top of the front parapet. The level of floor of under-sluices is R. L. 857·00. The head sluice or regulating bridge consists of thirteen bays, each of 21 feet span. Plate No. 61 shows in detail the various existing works.

Originally these 21 feet span vents were each subdivided into three sluice openings of five feet span each; thus the Head Regulator had thirty-nine sluice vents, each of five feet span. These openings were closed by timber draw-gates, worked from above, by a traveller running on rails, laid on the parapet; these gates, when raised, were hung from single hooks fixed in the face of the regulator above the vents. There were two sets of grooves in the piers, the front one being used for timber baulks or karris when the gates needed repairs. Baulks were sometimes left in these grooves in order to only take off

the top supply into the Canal, and not that at floor-level of regulator vents which was placed at R. L. 859·00, or two feet above floor-level of undersluices. Plate No. 62 shows the regulator, with the vents of five feet span, as it was originally constructed, and Plate No. 63 shows the travelling winch for lifting and lowering the gates at that time.

We thus in the early years of the Canal find that it was found necessary to use timber baulks as a raised sill, to take off the upper and exclude the lower level heavily silted water. This, then, may be taken as the nucleus for the permanent later developments.

The full supply of the Canal as designed was 9·43 feet on the Garhi gauge, which is situated twenty-six miles below the head, and the corresponding discharge was 6,000 cubic feet per second. In 1891-1892 this gauge registered 7·77 feet with 4,500 cubic feet per second passing down the "Main Line." Irrigation was developing in the lower reaches of the Canal, and the demand increasing while the head reach of the system was silting up to such a serious degree as to become alarming.

Colonel J. W. Ottley, R.E., C.I.E. Chief Engineer, Irrigation Branch, Punjab, in his notes of the 24th January 1893 on "Reports regarding silt deposit in the Main Line, Sirhind Canal," laid down seven works, partly of a remedial and partly of a preventive nature, as being necessary to ensure greater control over the silting, which was then going on in the main line. This paper deals with the seventh proposal, which was to obtain more water-way through the head than was then available, and this it was proposed to do by the addition of a new length of regulator with a sill at R. L. 867·00, that is, eight feet above Canal-bed and 10 feet above floor of the undersluices, thus supplying the Canal with water taken off the top of the stream or pond in the river instead of that at the lower levels.

In considering this question, the first point which presented itself to Mr. Reid, the then Executive Engineer, was "that the system of working which it was proposed to apply to the new regulator must also apply to the old regulator." The regulator consisted of the thirty-nine vents, in thirteen Main Bridge Arches; each bridge arch was of 21·0 feet span, broken up by three openings each, 5·0 feet wide and two jack piers each 3·0 feet thick. Jack arches closing the face of the regulator sprang at 9·0 feet from Canal-bed; they had a rise of 0·67 feet, and thus the openings into the Canal were permanently closed at about 9·4 feet on the average, above bed of Canal. The available water-way was then 39×5·0 feet, or 195 lineal feet. Silt baulks were kept permanently fixed in these vents, to a height of 4·0 feet above regulator floor; thus the available water-way was 195 feet×9·4′—4′·0 or 1,053 square feet, and through this section a discharge of 6,200 cubic feet per second had to be passed to give 6,000 cubic feet per second at the Garhi gauge. Now to do this it was necessary to head up the stream to 13·0 feet above Canal-bed, merely to allow the supply entering the Canal to run over the heavy silt deposits just below the regulator, and not to obtain head to drive the required discharge into the Canal. It was found in practice that only 0·3 feet to 0·5 feet head was sufficient to give the required velocity necessary to pass the discharge, and no trouble was experienced beyond that of raising the river surface for the sake of over-topping the silt in Canal; besides, the supply was invariably syphoned into the Canal, as the Canal-supply was rarely below 10·0 feet above bed, and the top of the openings was, as before stated, 9·0 feet above bed. This arrangement was not looked upon with favour, and the advisability of raising the vents into the Canal was suggested.

The advantages to be gained by raising the openings into the Canal were recognised, but exactly the same conditions of working would obtain, if the openings were closed at 13·0 feet above bed, and their sills raised to about R. L. 867·00, or 8·0 feet above Canal-bed. The disadvantages were that the river would be kept headed up to suit the raised vents, consequent loss of water by leakage of sluice-gates and shutters, percolation under the weir due to an increased head, and an undue strain on all parts of the river works.

This difficulty and the disadvantages enumerated were partly overcome by Mr. Reid's movable sill, enabling the height at which water could be

taken in to vary within limits. He proposed to take down the jack piers then existing, and to throw open the main bridge-arch openings entirely above R. L. 866·00 and to work the regulation by means of gates 21·0 feet wide in the clear. The advantage gained in lineal length of water-way alone would be 13 × 21·0 feet, or 273 feet, in place of the 195 feet which up to then was in force while the jack piers remained. The regulator would have a permanent solid masonry sill, raised to R. L. 866·00 or 7·0 feet above Canal-bed, and behind this permanent masonry sill would rise and fall a shifting sill, consisting of a series of regulating gates 3·5 feet in height, which when not in use would not be hung in the manner that was usual, from the tops of grooves in which they would work, but were housed at the bottom of their grooves, in recesses made for their reception on the downstream side of the permanent sill, built of solid masonry. This shifting sill was able to be lowered below the level of the top of the permanent sill. At that time it was sufficient for the shifting sill to rise and fall a maximum of 2·0 feet, but the height of these sill-gates was made 3·5 feet, as at the time it was considered advisable to do so, in case it was subsequently found necessary to raise the weir crest or height of shutters. As need arose, owing to silt-accumulation in the Canal, or still pond on the river-side, this shifting sill could be raised up to the required maximum height of 9·0 feet above Canal-bed, that is to R.L. 868·00. The maximum Canal-supply then was 6,200 cubic feet per second at the head, and this was proposed to be taken over a sill 9·0 above Canal-bed or at R. L. 868·00, with a velocity of 5·68 feet per second; the surface of the stream was 13·0 feet above bed of Canal, and thus 4·0 feet of the top water was drawn off.

The reasons for the shifting sill being fixed at 9·0 feet as a maximum and 7·0 feet as a minimum above Canal-bed were—

(*i*) that at certain seasons a sill 9·0 feet above floor would be inconveniently high. It was not safe to head up the river-supply to 13·0 feet above bed of Canal till after the 1st of October when the flood season had passed; while during the flood season the river-water was kept headed up to 12·0 above Canal-bed; and during the same period the sill-level was kept ordinarily at 8·0 feet above floor of regulator or Canal-bed.

(*ii*) The minimum height of sill was that which would suffice at the end of the rabi, when the supply in the river and the silt in the Canal were both at or near a minimum. The minimum supply in 1891-1892 occurred on the 8th March, the Garhi gauge then recording 6·19 feet with a corresponding discharge of 3,161 cubic feet per second, which included every drop of water available from the river, the river gauge on the down-stream side of the regulator at the time reading 9·05 feet. At such times of short supply it was not desirable to hold up the available river-supply more than was necessary as the greater the head the greater the loss by percolation under the weir and by leakage of the shutters and under-sluice gates. Under these conditions the minimum height of sill was fixed at 7·0 feet above bed, over which the required supply could be passed with 9·5 feet on the river-side of the regulator, which was not considered excessive.

Other works for relieving the silt in the Main Line of the Canal were just then in course of design which made it very probable that the maximum Canal-supply of 6,200 cubic feet per second would be passed over a sill 9·0 feet above bed of Canal with a less depth than 4·0 feet.

The permanent masonry sill has a vertical face on the river-side, while on the downstream face it is bellied to suit the fish-bellied shifting sill-gates working immediately behind it. The shifting or movable sill slides vertically in a groove of its own as shown in Plate No. 63. In order to ensure descent as well as ascent it is lowered and raised by a push and pull-bar attachment, primed on each end of the gate. The push and pull bar consists of two parts, the lower is an I steel beam, 4 inches by 3 inches in section; the lower end of this I beam has its web rivetted to the priming which is a bolt, passing through a conical opening in the top of the gate; its upper end is rivetted by

its flanges to two flat bars respectively. These flat bars are continued by a $2\frac{1}{2}$-inch diameter screwed shaft which is rotated by means of a horizontal bevel pinion in a cast iron cap bolted on to the top of the groove. On the screw shaft rises and falls a gunmetal die, which has trunnions on two of its vertical faces; it also has a female screw in which the screw shaft works. The upper ends of the flat bars of the push-and-pull attachment are eyed, and these eyed ends are fitted on to the trunnions on the gunmetal die. The outer edges of the shanks of the grooves are sufficiently thickened to prevent the die from dropping out of the groove and so permits it to have but a vertical motion; the groove thus forms a slide-box, while the die with the eyed ends of the flat bars forms a slide block, and by making the screwed shaft revolve in a cap bolted on to the groove the groove itself becomes the anchorage which is required to enable the sill to be depressed against obstruction. To enable the sill-gates to rise and fall equally at both ends (there being two grooves and two such gear as already described, one at each end of the gate) it was necessary that both screwed shafts should rotate with equal velocities; this was obtained by having a vertical bevel wheel at either end above the parapet, each in gear with the horizontal bevel pinion above the grooves. These vertical bevel wheels are connected by a horizontal 3-inch diameter shaft, which is supported by two cast iron pedestals along its length, these pedestals resting on the other girder to which they are bolted. The horizontal shaft is rotated by a couple of ratchet arms fixed permanently to it. The ratio of the wheel to the pinion at the top of the shaft is about 2 to 1; thus the sill-gates are lowered or raised equally at both ends by an intermittent vertical motion. The arrangement is compact and inexpensive. The screws are in a position in which it is hardly possible for them to be injured, while at all times they can be cleaned and oiled without difficulty, while the moving parts of the gearing are never below water-surface.

Practically all regulatoin, other than that done by the under-sluices and shutters, is done with these lower or sill-gates, which are raised and lowered throughout according as it is desired to decrease or increase the supply entering the Canal.

A second set of gates 6·25 feet in height is also provided for the purpose of shutting off the upper portions of these large vents during Canal closures or when high floods are on. These closing or upper gates are hung when not in use from a suspender placed above and midway between their own grooves. By a knock on the lever the gates are freed and descend in a few seconds; this method is only employed if the floods come down before the usual ordinary lowering by the travellers is effected. The grooves of the upper gates are placed immediately downstream of the regulation or lower sill-gate grooves, and these gates are lowered and raised by means of an ordinary traveller running on rails laid overhead. This traveller is shown on Plate 6. The bottom of these upper gates is 8 feet 10 inches above floor of regulator, thus lapping the sill-gates 2 inches, the opening between the two being closed by a baulk of timber 5 inches × $3\frac{1}{2}$ inches section, bolted on to the river face of the upper gate. In closing the Canal completely it is only necessary to first raise the lower or sill gate, closing the opening to 9·0 feet above bed, and then lowering the upper gates on to their seats, which consist of teak blocks bolted into the masonry.

In adopting this scheme of remodelling the regulator the width of the roadway between parapets was reduced from 18·0 feet to 15 feet 5 inches; this contracted width afforded ample room for the regulating staff, while there is no traffic to speak of over the bridge; the narrowing was due to the setting back of the face-wall of the regulator rendered necessary by the double grooves, and by the room taken up by the permanent masonry sill, the cut-waters of the piers being built up to carry an outside girder on which and on the parapet runs the ordinary traveller before referred to.

This scheme was estimated to cost as follows and the amount was debited to the open capital account of the Canal :—

	Rs.
I—Works	56,286
II—Establishment	12,946
III—Tools and Plant	844
Total .	70,076

A more detailed abstract of the estimate is appended to this paper, showing the quantities and headings under I—works. The scheme was executed during the cold weather months of 1893-94.

The subsequent improvements to the scheme were—

(i) Advancing the river face of the permanent masonry sill, by an addition of 4·0 feet to the thickness of the wall and so at the same time strengthening it. The advantage to be gained was to better ensure the face of the regulator, being swept clean during the periodical scourings of silt from the undersluice channel by means of the undersluices; at the same time the 50 feet width of stone-pitching in front of the regulator was grouted. This was carried out in 1895.

(ii) In working the sill gates it was found that it took six men 29 minutes hard work to raise them 2·0 feet, that is, from 7 feet to 9 feet above regulator floor. It was calculated that, with two gangs of men, it would take three hours and forty minutes to completely close off the canal when the sill gates were down and a freshet came on, and it was thus, only after the closing of the canal had been accomplished, that the regulating men could be put on to the undersluice travellers. The slow movement of the lower or sill gates took up most of the time, and was remedied by substituting $1\frac{1}{8}$ inches pitch, double, $\frac{5}{8}$ths inch thread, in place of the $\frac{1}{2}$ inch pitch, single, $\frac{1}{4}$ inch thread screws of the screw shafts. The sill gates now have a more rapid movement and at the present time 4 minutes are taken by six men to raise the sill gates from 7 feet to 9 feet, and working with two gangs of men the canal can be comfortably closed off in an hour and 50 minutes. This modification of the screw shafting was effected in 1900-01.

The object of the permanent masonry and the sliding sill was to keep out of the canal the heavy silt in the lower depths of the river water. In front of the regulator there is a comparative still water pond, formed by the closed undersluices and a divide wall separating the weir from the other works. This pond has its bed paved with concrete blocks 4 feet by 4 feet by 2 feet in depth, and soundings of silt in this pond, otherwise the undersluice channel are regularly taken; when the soundings show that the silt on the channel bed has risen to about 4 feet, the canal is closed and the silt is scoured out through the undersluices. Thus it will be seen that the anxiety caused by the silting of the Main Line of this canal has disappeared and that methods, based on correct scientific principles, are now in force to keep silt out of the canal. The Garhi Gauge is now limited to 11·5 feet with a corresponding discharge of 8,000 cubic feet per second, this increase having been effected without modifying the regulator in any way, after the works described in this paper were executed; while to obtain this increased discharge the water surface in the river has been headed before the regulator up to R. L. 871·00 or 12 feet above canal bed and the anticipated advantages have been fully realised.

PUBLIC WORKS DEPARTMENT, PUNJAB.

IRRIGATION BRANCH.

(*Project*).—Sirhind Canal.

(*Canal or Branch*).—Headworks.

(*Name of work*).—Remodelling Rupar Regulator at the head of Sirhind Canal.

Fund (*Major Head*).—Imperial [illegible]—Capital.

(*Departmental Head*).—I—Works.

(*Minor Head*).—(I) Headworks.

(*Sub-Head*).—D.—Regulators.

No. or quantity.	Details.	Rate Rs. A. P.	Per	Amount Rs.	Totals Rs.
19,?32 c. ft.	Dry stone	[illegible]	[illegible]	[illegible]	
19,0?? „	[illegible]	[illegible]		[illegible]	
4,350 „	[illegible]	[illegible]		[illegible]	
277,0?? „	Earthwork	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]		[illegible]	
2,?57 rft.	[illegible]	[illegible]	[illegible]	[illegible]	
??? c. ft.	[illegible]	[illegible]	[illegible]	[illegible]	
??? ft.	[illegible]	[illegible]	[illegible]	[illegible]	
??? lbs.	[illegible]	[illegible]	[illegible]	[illegible]	
13 c. ft.	[illegible]	[illegible]	[illegible]	[illegible]	
680 lbs.	[illegible]	[illegible]	[illegible]	[illegible]	
2 No.	[illegible]	[illegible]	[illegible]	[illegible]	
??? rft.	[illegible]	[illegible]	[illegible]	[illegible]	
					[illegible]
	Add [illegible]	[illegible]	[illegible]	...	[illegible]
					[illegible]
	II—[illegible]	[illegible]	[illegible]	...	[illegible]
	III—[illegible]	[illegible]		...	[illegible]
				Total Rupees	?,??0

PAPER No 38.

On the best value of Kutter's "N" to adopt in Canal design.

About ten years or so ago orders were issued in the Punjab that N was to be taken as 0·025, though even at that time there were doubts if this was advisable. No regular statistics of this figure have been kept and though the value of N is usually worked out for the more important discharges observed from time to time, yet I am doubtful if any good result would accrue from collecting these data; they vary very much for the same channel, and even for the same site, and are not in all cases reliable. However, I think we know enough now to fix within fairly narrow limits what N really is on our canals. About ten or twelve years ago I carefully noted about 20 or 30 actual values of N, as observed by myself on the Bari Doab Canal, and though I did not keep the details I remember that the average value came to 0·023; some few went up to 0·025 and some went down to 0·021. Afterwards on the Western Jumna Canal I found one or two cases where the higher limit 0·025 obtained and also some where N was 0·020. In one case it was 0·019, but here the bed was more or less scoured out, and I expect this may partly account for other similar cases we occasionally hear of. On a quite new and well dressed channel, however, there is no doubt that N does go down to 0·020, and even lower, but in all probability this will not last very long, as the bed and banks get gradually irregular, even when there is no erosion to speak of. Wherever the bed was silted and the banks fairly well kept I found the value of N to be fairly steady at 0·0225, and for an all round figure for canal design I think this is the figure we ought to adopt.

PAPER No. 39.

Economy of water in Deccan irrigation.

The policy and methods described in this paper apply to new irrigation works in the Bombay Deccan and are based on a close study of the necessities of irrigation in this region. Nearly all the impediments to the progressive growth of irrigation arise from the want of a settled system in the distribution of water and the fitful character of the demand for water for irrigation of the staple grain crops of the country. The extent of irrigation of such crops and the revenue from them vary with the rainfall. When the rainfall is sufficient, there is little or no demand for canal water for the crops; but during years of drought or scarcity, the pressure on the canal water is large and at times intense, and a severe strain is put on the canal staff.

The principal irrigated crops in the Deccan may be conveniently arranged into three classes, namely:—

(1) Perennial and garden crops (principally sugarcane);
(2) Monsoon and rabi crops (chiefly staple food-grain crops);
(3) Eight months and hot weather crops (vegetables, ground-nut, fodder, etc.).

Crops of the last class are subsidiary and unimportant. Their area is always limited and the yearly variation is small.

Class (2) are the staple grain crops of the country like juar, bajri, wheat, etc., already mentioned.

The demand for water for crops of class (1) is fairly steady and grows steadily with the guarantees for a continuous supply and the regularity of the waterings. Sugarcane is the most important crop of this class whether regard be had to the regularity of the demand or the yearly produce and revenue.

The demand for water for crops of class (2) being variable, the water courses are often only large enough to meet the average demand, which is for 25 to 40 per cent. of the area commanded. Since every field near a canal does not require water, the area which a channel should command is enlarged, so that where all do not require water some may, and the demand may come up to the average. Cultivators ask for water for grain crops when they need, and the canal officer supplies water when he can. There is no touch between supply and demand. It is best to encourage the growth of crops like sugarcane for which there is a sustained demand year after year, after reserving enough of water supply for the average area of other crops requiring irrigation in normal years. The policy of utilising all or the greater portion of the available supply in a famine year for food grain crops and thereby disturbing the irrigation regularly available and destroying the credit of the canal, ought never to be encouraged. If a work is managed so as to get the best return from it every year, not merely as a reserve for the protection of grain crops in seasons of drought, a permanent industry will be provided to the cultivators, and the wealth produced in normal seasons will help to mitigate distress in times of scarcity.

For irrigation purposes in the Deccan, the year may be divided into three parts, namely:—

(1) The monsoon extending from 16th June to 15th October in which the staple food-grains like bajri, pulses, etc., are irrigated; (2) the cold season extending from 16th October to 15th February in which rabi crops like juar wheat, gram, etc., and eight months' crops like ground-nut are extensively grown; and (3) the hot weather from 16th February to 15th June. These three equal divisions of the year, each of four months, form definite and convenient periods or bases as they are called, for estimating the duty of water.

The first step towards orderly progress on every irrigation work is to find out the minimum supply of water available, and to determine what area of perennial, rabi and other crops may be irrigated by it as a permanency in each season, and in what proportion.

The canal should be divided into two or more divisions. In the one nearest the head which usually comprises more than half the length of the canal, irrigation may be carried on throughout the year. Perennial crops may be irrigated in this section. The second division or length of canal should be given water till the end of the rabi season (15th February) only. The third division should have water when available during the monsoon only.

Of the minimum supply available, a fraction should be set apart for irrigation till the end of the rabi season in the second or rabi section of the canal, and another fraction for uncertainties of evaporation which vary with the seasons. The balance which is usually $\frac{2}{3}$ to $\frac{4}{5}$ of the total minimum storage may be taken as a dependable supply every year. From this supply could be estimated the description and areas of crops for which water can be given regularly in each season from year to year. The next step is to distribute the areas of the various classes of crops in a recognized proportion to each of the villages under the command of the first division of the canal. For the villages nearest the head works, the area of perennial crops allotted should be just large enough to maintain a constant demand for them, but not too large to constitute a surfeit. Excessive facilities will lead to overcropping and impoverishment of the soil. The farthest village up to which perennial irrigation may be extended will depend on the possibility of maintaining a supply in the canal throughout the year without excessive loss of water in transit. The distribution of the area of crops to be irrigated, by villages, which is done after taking into account the extent of perennial irrigation at present carried on, should not ordinarily exceed $\frac{1}{4}$ or $\frac{1}{3}$ of the total culturable area of the villages under command.

Storage being very expensive in the Deccan, irrigation of crops which require water for 3 or 4 months of the hot weather only should not be allowed as a permanent arrangement.

All crops should be given water without stint in every monsoon in every village and in all parts of the canal wherever the canal is large enough to carry the requisite supply. In designing a canal and distributaries, this object should be kept in view and ample facilities provided for the purpose where it can be done without a disproportionate outlay.

Having disposed of the minimum water-supply which can be depended upon in every season, the canal officer should take special care to utilise to the utmost advantage by temporary measures, all the variable excess above the minimum, which cannot be permanently allotted to villages. The excess supply is usually due to storms in the months of October to December and from end of April to beginning of June. The excess supply in any particular year in the tanks on the Deccan plains, which do not regularly fill every year, may mean the difference between the storage at the end of the particular monsoon and the lowest supply in the lake in a series of eight or ten years. Arrangements should be made to deal with the distribution of the surplus supply very promptly; otherwise the superfluous supply might remain unutilised till the next replenishment. Usually, rabi crops and hot weather crops could be specially encouraged in tail villages. When the management of a canal on the lines indicated in this paper has made some progress, there should be no objection to sell water at reduced rates by a general notice in order to dispose of the supply, which would otherwise produce no crop or revenue and would, for all practical purposes, be wasted.

It may be well here to give some details of the system and measures recommended for the economical use of water—

(1) in reservoirs;

(2) on the main canal, and

(3) in distribution.

In Reservoirs.

A table of contents should be maintained for a reservoir for every tenth of a foot. It may be necessary to resurvey contours once in 20 or 30 years to make allowance for accumulations of silt in the interval.

The loss of storage by absorption, leakage and evaporation for any given depth of water in a lake is the difference between the total quantity consumed and the quantity delivered at the sluices for irrigation. By frequent experiments and by the aid of a correct record of discharges at the sluices, the loss by leakage and evaporation should be correctly determined month by month in the fair seasons, as well as, say, for every five feet of depth in the lake. The loss depends on the level or elevation of water, the season of the year and the character of the season.

The loss by evaporation should be observed and recorded on all occasions on which sluices are closed for canal clearances or repairs.

The quantity of water required for irrigation in each month should be calculated at the beginning of the season and a forecast table prepared showing the level at which water should be maintained on the first of every month between November and the commencement of the following monsoon. A recent practice in the district with which I am connected is to have two levels estimated for at the beginning of each month, one based on a strict estimate of the consumption of water and the other on a liberal estimate. If the strict estimate is worked to, without a reduction of the area and without complaints of short supply, it will bespeak careful management on the part of the canal subordinate who earns the commendation of his superior for successful management. When the liberal estimate is exceeded at any period, it is an indication that the danger point is reached requiring stricter supervision on the part of the canal officer.

The forecast is compared with actual results month by month and the duty of water for each month and season recorded. A correct record of the results of one year will lead to further economy in subsequent years and ensure a progressive growth of efficiency in canal management.

Copies of forecasts made in this way and the actual results obtained on the two lakes in my charge for the season just ended are available for inspection.

An automatic level recorder, supplemented by fixed gauges, should be maintained for observing the water level in each tank.

On the Main Canal.

The main canal should be divided into a convenient number of sections usually varying from 15 to 30 miles. The discharges at the head and tail of each section (and at several intermediate points varying with the importance of the canal and the value of water) should be observed twice or oftener daily in the fair season, in order to determine the total quantity of water consumed in the section, the quantity delivered into the outlets and that lost by waste or leakage in the section of the canal. An important canal official should be responsible for the accurancy of the gaugings and the correct working of the modules. The official responsible for these should be a different person from the one responsible for the utilization of the water issued at the outlets. The latter should account for all the water supplied at the outlets for irrigating crops in his charge. This will ensure that two persons will be interested instead of one in the accuracy and economy of the distribution at the outlets. By means of the telegraph line, such as has been newly laid on the Nira Canal, the canal officer can be apprised of the results of the working of the canal, from day to day.

The canal officer would make allowance for a reasonable loss by absorption, etc., in transit, but he should be able to prevent all wastage due to negligence or want of system.

The canal discharges are at present calculated by observing the surface velocity with the aid of floats and assuming certain coefficients due to the hydraulic mean depths, taken from tables. They are also calculated, though on rare occasions, by taking longitudinal and cross sections of the canal waterway.

It is proposed to set up at two or more points in each canal section an automatic float recorder and a current meter similar to those extensively used

in the United States. One set of these instruments has recently been ordered for the canals in my charge and if, as is expected, they give satisfactory results, it is intended to use two or three sets of these instruments in each canal section.

The canal official in charge of the section should also be required to submit at the end of every ten-day or other period a report giving the quantities of—

(1) total supply of water consumed in each section;

(2) total supply issued for irrigation at the outlets, and

(3) total supply lost by leakage, absorption and evaporation, with explanations for all important variations.

By comparing the results of one section with another and the results achieved by one official with another, there will gradually be determined the quantities of water lost by leakage and absorption for the varying depths or discharges in the canal. These definite results will reveal the weak points on the canal where the leakage is excessive and will eventually enable remedies to be applied at the proper places.

The lengths of canals and distributaries where the leakage is found to be excessive might be puddled or made staunch by concrete or by other methods usually practised.

It will be necessary to keep all outlets running full for a limited number of days in every ten-day rotation, instead of leaving open a large number of distributaries at a time and allowing water to dribble into them. The loss by absorption in the latter case is enormous. The best practice is to shorten the period in which an outlet is kept open by sending down a large volume of water. This will necessitate all or a number of outlets in each section being kept closed for a certain number of days in order to maintain water at a high level to run other outlets full in the same section or other sections. The number of days the outlets are to be opened in each section, the level at which water should be maintained by movable stop-gates or otherwise and the number and locality of the particular outlets which have to be kept open to receive the excess flow in each section, must be determined by trial and according to local conditions by the responsible canal officer.

The supply of water at the tail of a section will not usually conform exactly to the table of regulation. There will always be some small variations. At the end of a section, a few outlets should be kept open to utilize the surplus water which arrives irregularly. These "irregular" outlets will water the crops dependent on them once in ten days but not on fixed dates. The waterings will be given from the tail of the distributaries upwards, and whatever area is left unwatered at the end of the ten-day (or other) period would be supplied by special arrangement on the last day of the expiring rotation or the first day of the next.

In Distribution.

I have already indicated how the distribution of the areas of crops which can be permanently irrigated year after year is to be made by villages.

The area allotted to each village is next distributed by outlets in proportion to their discharging capacity. Wherever the small size of the outlets and channels interpose an obstacle to the equitable distribution of the areas, the outlets should be enlarged gradually, say in the course of two or three years after a permanent scheme is evolved.

Since under the conditions of the Deccan, only a part of each village can have irrigation and not the whole, it is important to induce the cultivators to concentrate the whole area allotted to the village in a number of blocks situated in selected soils, each block measuring, say, from 60 to 300 acres. For areas so distributed the water may be given on a long lease, say, of six years at a time. The cultivators will be required to carry on a rotation of crops within the blocks. This will obviate the great loss of water due to

the network of distributaries scattered over the whole area, which in the hot weather extends at times over 30 times the area actually under crop. Definite proposals for the concentration of irrigation in blocks have been submitted under the designation of the block system. The system is at present being introduced on the Nira Canal under the orders of the Government of Bombay. The cultivators were at first reluctant, but after the aims and objects of the system were explained by a Committee of Government officers specially appointed for the purpose, they have given their consent and are at present co-operating in the extension of the system.

The area of crops under each outlet being thus fixed, the next step is to supply water in proportion to that area. This can be done only by a reliable module. After much consideration, a module has been designed which is described in a separate paper read before the Conference.

The distribution of irrigation in the blocks and the correct measurement of the quantity of water by module introduces two valuable elements of fixity in irrigation of which the conditions in the Deccan sorely stand in need. With the definite information and data which these measures supply, it would be easy to determine leakage and waste and apply remedies so as to raise water distribution gradually to a high standard of efficiency. A gang of watermen may be employed to go round the distributaries, say, once in two months, and give one watering to all crops under an outlet and observe the discharge and the period required for the watering. From the results of their work, the duty of water can be definitely fixed for a term and the cultivators and local watermen held responsible for the intermediate waterings. Complaints can be verified by the aid of the same gang of watermen. The number of days for which an outlet should be kept open will be determined by the results of the last distribution by the gang of watermen referred to. The time allowed for the intermediate waterings which have to be done by untrained men should at first be 10 or 15 per cent. more than that actually taken by the trained gang.

Wherever the block system is not introduced, it is necessary to prepare and maintain share lists showing the area of crops which may be allotted to every cultivator who may apply for the use of canal-water. The share lists may be prepared once in five or six years on some approved principle, the distribution being made in an equitable way, care being taken that the areas allotted to the regular customers do not undergo large fluctuations and that no new applicant desiring to participate in the benefits of the water-supply is kept out of the share lists without satisfactory reasons. Cultivators should be encouraged to combine and form temporary blocks by offering water to them at reduced rates.

Lists should also be kept ready giving the names of *villages* which should receive the surplus water due to savings after the fixed areas are provided for, and also the savings and surplus water due to rain storms after or before the regular monsoon. The total area which can be irrigated will be uncertain, so the lists will only show the percentage of available area which a village will receive and the circumstances and conditions under which the supply will be given.

The distribution to individual cultivators may be effected in a fixed proportion to the area under command owned by each or on some other approved principle.

The waterings should usually be given from the lowest fields upwards and as soon as the field nearest the canal is reached and watered, the outlet should be closed.

In the rabi season and the hot weather, outlets should usually be open for 10 to 12 hours during day time only. The practice on the canals in my charge is to hold the canal inspector responsible for watering about four acres of sugarcane per cusec delivered at the outlet in 11 or 12 hours of the day. The duty varies occasionally from three to five acres according to the distance from outlet of the area watered. Tables of discharges are maintained after actual gauging for various heights of water as observed in a straight reach at the head of the distributary channel.

In order to keep touch with cultivators, a Check Inspector appointed from the cultivators' class moves about from village to village, collecting complaints where there are any and transmitting them direct to the Subdivisional Officer.

Inspectors and watermen are rewarded at the end of each of the three seasons into which the year is divided, according to the increase of efficiency and economy of water distribution in their charge.

If the arrangements indicated above are carried out and the results tabulated and recorded from year to year, the working of any one tank or canal can be compared with all others similarly situated, and the methods which have worked well in practice will gradually find general favour. What is most important to arrange for is definite methods of working scientific observations for determining the quantities of water and a clear understanding with cultivators regarding the regular use of the water; and, generally, to ensure continuity and method in all canal operations.

The work in this connection should be subdivided into distinct subheads. The work on each subhead should be clearly defined, the person responsible should have clear instructions, and the results collected, tabulated and recorded for each season and year. This appears to be the secret of successful management.

The above methods and suggestions are based on the experience gained in the endeavour to systematise irrigation operations on two of the largest canals in the Bombay Presidency excluding Sind. Most of the recommendations made in this paper are being practised on the two canals referred to with the exception of the block system and the use of the module. As already stated, the block system has been sanctioned by Government and is being introduced.

The module has been experimented with and estimates are under preparation for obtaining sanction for its extended use.

The productive capacity of the works and the agricultural efficiency of the tract served by them will improve with the improvement of the methods employed to ensure economy in the use of the water. The demand that the measures here recommended make, is precision in all details and a continuity of policy for a series of years. The policy will involve a small but inappreciable capital outlay on puddling and stopping leaks and on the equipment of modules. There will be no addition to the recurring expenditure, there may even be a reduction of charges on establishment, and the canal officer will be saved the unending worry of a daily adjustment of water-supply to the daily varying demand.

A close watch must be kept on the expenditure of water in the reservoir basin, along canals and channels and in the application to fields. With a sustained effort and by scientific methods, it ought to be possible to keep an account, correct for all practical purposes, of the receipts, issues and expenditure of water in each of these stages. A certain amount of loss is inevitable; but the engineer should see that the loss is reasonable in each case, and there is, so to say, no defalcation anywhere.

PAPER No. 40.

Areas debarred from Canal Irrigation.

The principle of debarring areas from canal irrigation is no new one and has been enforced in various cases in order to avoid interference with established irrigation from wells or other sources, to guard against the rise of spring level and to prevent the use of canal-water in unsuitable soils. In general in these cases whole tracts have been so debarred and the prohibition ensured by not carrying distributary or watercourse channels into such areas.

In the project for the Fatehpur Branch of the Lower Ganges Canal a development was introduced in the extension of the principle both to whole villages and portions of a village. To quote from the note of the Inspector General of Irrigation, on the project:—

"One of the main features of this project is the proposal to debar from canal irrigation villages which are already sufficiently protected by irrigation from existing sources and to avoid interference with existing irrigation. It will readily be allowed that this principle, which is everywhere desirable, was an absolute necessity in the case of this project where the supply of water is so limited in comparison with the area under command and that the attention paid to the subject was fully justified. It was further proposed in the case of villages irrigating both from wells and from the canal, eventually to debar the "Chaks" dependent on wells."

In making these proposals three essential factors were postulated:

First—That the function of the canal was to protect the area under command; "protection" being defined as that supply of water which in famine years will make up the deficiency in the rainfall. To ensure protection it is necessary to provide for that percentage of the *malguzari* (cultivated and culturable) area in a village, the assured irrigation of which will in times of scanty rainfall render the village self-supporting and able to pay its revenue demand. This is the "minimum percentage."

Secondly—That it is necessary to fix a "maximum percentage" or the percentage of area in a village which may be irrigated without injury to the interests of the most improved systems of agriculture; and from which would be determined for purposes of distribution the limit which should not be exceeded by canal irrigation under any circumstances.

Thirdly—That in both these limits there must be absolute non-interference with existing sources of assured irrigation.

With these preliminaries, the limits to be adopted were considered with regard to the conditions of various districts in the Provinces.

After some discussion, the Local Government accepted the following as the basis to be adopted in the distribution of canal irrigation on the Fatehpur Branch and the Ghatampur Distributary Extension which also serves a part of the same districts:—

(*a*) That calculations should be based on the cultivated, not the *malguzari* area of a village.

(*b*) That permanent stable wells should alone be accepted as an existing source of assured supply.

(*c*) That a village should be entitled to claim canal irrigation up to a minimum of 35 per cent. of the cultivated area, less the area from permanent wells.

(*d*) That the maximum area to be irrigated should be fixed at 45 per cent. of the cultivated area.

(*e*) That all lands irrigated from permanent and reliable wells should be absolutely debarred from the use of canal-water.

(*f*) That the calculations of area should be based on the records of a series of ten years.

It will be noted that the application of the principle of debarring to the Fatehpur Branch was in the first instance necessitated by the insufficiency of the canal-supply for the area commanded. Now the original project contem-

plated the introduction of the canal to command a much larger area than is served by the completed scheme. This extra area has in fact been debarred because of the existing large percentage of irrigation from reliable wells.

The distribution of water has in actual work been calculated on the base of 40 per cent. of the cultivated area, *plus* a percentage of the culturable area, *less* the area already irrigated from permanent wells.

The percentage was adopted as a mean between the limits fixed by Government, and with the smaller area under command in the modified scheme it was a more convenient base. It was necessary to make an addition for the culturable area, because the districts served were not in a highly prosperous condition and in some cases large areas of good soil were out of cultivation. The result has justified the measure; in the two parganas of the Cawnpore District served by the Fatehpur Branch and the Ghatampur Distributary there has been an increase of over 12,000 acres of cultivation since the introduction of the canal. In the Fatehpur District the advance has not been so great, but the development of the canal has there been much slower due to the ignorance and apathy of the cultivators.

To fix the "debarred" areas, lists of all fields irrigated from each well during a period of ten years were collated from district records and the fields marked off on the large Irrigation sheets. It was then necessary to examine practically every well as numbers of so-called permanent wells were found to be unstable. The area to be debarred was fixed at $2\frac{1}{2}$ times average annual irrigation on each well and this agreed fairly closely with the area shown by the lists as within the command of the well.

The irregularity of the well plots and the ownership of wells was a source of trouble. To simplify the working of the scheme and to avoid confusion with canal areas, it was necessary to keep the debarred areas as compact as possible and to take advantage of natural boundaries such as roads, etc., to define them. This raised an outcry from any individual who found his fields shut within a debarred area when he had perhaps only had an occasional use of well water.

Owners of wells raised very strong objections, possibly anticipating that the acceptance and recognition of the debarred area on any well would create a prescriptive right for others to the use of the well where they had previously only been admitted on sufferance.

The scheme has not been successful in operation. In providing for the canal irrigation it has not been possible to avoid taking canal water-courses into the neighbourhood of well plots; and in such cases the water has been diverted to the well area. This was to be expected, as the well area is in good soil and the best manured land in the village, while the use of canal-water saves labour and outlay. The penalty of the double water-rate imposed under the Canal Act is not a complete deterrent, for even with this rate the cost of the crop is less than with well water.

It would doubtless be practicable so to use the powers conferred by the Canal Act as to make irrigation of these lands by canal-water so troublesome and expensive that cultivators would revert to the use of their wells; and it would be judicious if the area were large.

On the Fatehpur Branch, however, the well areas are comparatively small, the plots are scattered; and the resultant gain from the maintenance of wells would not be large. Nor is there any guarantee that, even under a condition of efficient debar, the wells would be maintained. Where areas are small, cultivators might abandon their wells and take entirely to canal irrigation, reserving the debarred lands for irrigated kharif crops.

In the Sah pargana of the Fatehpur district, in which well irrigation was probably more general than elsewhere, the area irrigated from permanent wells was only 7·5 per cent. of the cultivated area and the highest percentage in the pargana is 16.

In no single case on the whole system can an entire village be debarred under the rules. Indeed the rules create a dilemma; on the one hand the village is entitled to claim a minimum percentage of irrigation in its area; on the other the percentage cannot be given without an interference with

established irrigation. Though on the whole the requisite percentage has been given—there are many cases in which expansion of canal irrigation has been interfered with by the attempt to adhere to the rules.

For the first 35 miles of its course the Fatehpur Branch runs on a narrow watershed between two deep rivers, thereafter the main line and the terminal channel run along the high land skirting the Rind and Jumna rivers. With the exception of the two areas noted below the drainage outfall is ample and there is no danger of an excessive rise of spring level.

In the central tract in the Sah and Ghazipur parganas the drainage line to the north is undeveloped and consists of a series of depressions which in times of heavy flood spill from one to the other. Here the spring level is at present from 30 to 45 feet below the surface while on the south it runs to over 60 feet.

The second area lies in the Allahabad district to the north of the terminal line; here the drainage line to the north may require to be opened out, here too there are large flat depressions. Complete figures for the spring level of the whole of this tract are not at hand. The spring level appears to vary between 38 feet and 11 feet below ground level. In this second tract the wells are fewer and of small capacity. On the whole the maintenance of the wells that exist would have but little effect on the spring level of the country.

In the area served by the branch, the maintenance of the small well system in conjunction with a canal system does not appear essential to the welfare of the districts. The canal-supply will with an improved duty (and it is improving) be sufficient to ensure every village its maximum percentage. To ensure the maintenance of the wells, a very stringent system of debar must be upheld and this will be troublesome and may be purchased at too dear a price with the introduction of another means of oppression by subordinates.

It is possible that with increased prosperity and expansion of cultivation, which the canal system cannot provide for, villagers will of themselves keep up their wells at all events for special crops such as opium.

The general policy of maintaining the existing well irrigation is admirable, but if it is to be effective on the introduction of canal irrigation, the unit of debar must be the village and not the minor area on each well.

PAPER No. 41.

Mr. Keeling's Gate.

This set of gates was designed to suit the condition of the lower reservoir which has been proposed for the Periyar Project Extension Scheme. Briefly put, the extension scheme aims at diverting and impounding the surplus which now passes over the escape of the Periyar reservoir. The impounding is proposed to be effected by increasing the capacity of the existing upper reservoir and by creating a new reservoir at the foot of the hills through which the tunnel now passes the water drawn from the existing reservoir.

The lower reservoir has a twofold object, *viz.*, increasing the storage capacity of the project for irrigation purposes and rendering it possible to obtain continuity of flow in order to generate electrical energy without wasting water. It is thus obvious that it must be possible to empty the lower reservoir at least once a year, *i.e.*, before the irrigation closure. The ultimate maximum discharge required during the period of irrigation pressure is estimated at 3,000 cusecs for purposes of calculation. The gates must be capable of discharging the required maximum under the full head or a low head. To avoid expensive channels and land acquisition the length of the dam for the location of the gates is limited. The conditions briefly-recapitulated are:—capacity for obtaining full discharge between 150 and 12-foot levels within a length of 180′.

The model, a photograph of which is given on Plate No. 2, represents one of the 4-foot gates. There are many details which it is impossible to reproduce in a model of this description without going to undue expense. The model is, therefore, to be taken as only exhibiting a general idea of the method proposed for working the gate.

When the sluice is closed the seating surface, which is left on the gate, is in contact with the surface of the frame of the opening and forms a continuous joint in one plane which is watertight without the assistance of stanching ars or any loose parts. In this position all the gear at the back of the gate is accessible for inspection. The lining of the opening will be formed of strong steel frames having a stiff boiler plate cover bolted, which can be removed and the chambers are of such a size as to permit of any of the rams or cylinders being withdrawn when the gate is closed. The rollers and their bearings can be inspected at the same time and the spindles withdrawn and replaced if necessary. This makes gunmetal bearings and journal lubrication feasible and their use will reduce the coefficient of friction and the effort required to raise and lower the gate.

To raise the gate pressure is first admitted to the main cylinder actuating the side beams, the rams are then forced out raising the gate off its seat and bringing the way on the side beams into line with the ways above the frame so as to give a continuous travel for the gate which is then in a position to be raised by the lifting cylinders in the chamber above pressure is admitted to the lifting cylinders and the gate is hoisted into the recess.

The distance piece between the cross-head connecting the rams of the main cylinder at the back of the side posts and the movable beam are of large diameter and have ample bearing surface to carry the weight of the cylinders and of the movable beams which parts are so arranged as to practically balance each other. Way is provided between the moveable beams and the side posts for the escape of silt the outlet being at the bottom.

The lifting rods pass through stuffing boxes into the chamber containing the lifting cylinder which are accessible and can be repacked at any time.

For the purpose of repacking a rope passes over a crab with a balance weight attached at the other end which is sufficient to balance both ram and cross-head so that the nuts on the cross-head can be slacked off and the rams

raised or lowered by hand. The rope and crab will be sufficiently strong and the latter will be fitted with a ratchet and pawl attachment to obviate the necessity for retaining pressure on the lifting cylinder while the gate is raised.

These gates are proposed to be operated by a motor-driven hydraulic pump and accumulator. In situations where motor-driven pumps are not possible a Diesel oil engine could be substituted or under certain circumstances a hand pump.

The hydraulic supply and exhaust mains will be laid on the top of the dam with a separate branch and hydrant for each gate. The requirements for each unit are then one hydrant and three valves —

(1) for main cylinders of side beams;
(2) „ „ lifting cylinder;
(3) „ „ lowering „

The return cylinders of the side beams are open to constant pressure when the hydrant is open to the main, thus making the return of the side beams to their seating automatic as soon as the exhaust of the main cylinders is opened. The side beams should be forced out during the time the gate is being raised or lowered, locking gear is therefore provided making it impossible to raise or lower the gate unless the main cylinders operating the side beams are first open to pressure.

Suitable stops are provided for all the movements of the hydraulic gear so that in no case can the stroke be overrun.

The weights given are based on the assumption that all the cylinders and working parts should be capable of taking an overload of fifty per cent.

The designers say that it would be simple to make an attachment to the underside of the gate to engage with an attachment on the side beams in such a way as to insure that the return rams of the side beams would draw the gate home to its seat as soon as the main cylinders of the side beams were open to exhaust, thus making the return to the seating independent of the water pressure.

As regards the question of wear due to high velocity the opening can be easily lined throughout with boiler plates in such a way as to ensure easy renewal.

The total weight of the 4 foot gate withs lining, etc., is approximately 78 tons and a rough estimate of the cost of the set of gates including all piping and connections and a motor driven hydraulic pump and accumulator amounts to £10,500 f. o. b.

The design embodies the conditions essential to success in such problems, *viz.*, simplicity, strength and accessibility of all working parts. The whole of the hydraulic gear has been designed by the West Hydraulic Engineerning Company of Luton, Beds, to whom the writer of this note is much indebted This firm has designed and erected hydraulic gear of all kinds for various Governments, among their most recent work being a great deal of the hydraulic gear for the cordite factory at Wellington.

G. I. C. P. O.—No. 912 R. & A., C. W.—8-8-1905.—650.—C. T. etc.